12

Hair, Christopher

From: Wilkins, Lynnea
Sent: Friday, January 06, 2012 11:02 AM
To: Hair, Christopher
Subject: Fort Calhoun And Cooper Acknowledgement Letter and FRN
Attachments: ML1200300270.doc; ML1200300223.docx

Chris,

As discussed, Kristy is logging these in now (letter -ML1200300223, FRN - ML1200300270)

Thanks Again!
Lynnea

Lynnea Wilkins, Project Manager
Fort Calhoun Station, Unit 1
Cooper Nuclear Station
Plant Licensing Branch IV
Division of Operating Reactor Licensing
Office of Nuclear Reactor Regulation
US Nuclear Regulatory Commission
Phone: 301-415-1377

A28

Mr. Thomas Saporito
Senior Consulting Associate
Saprodani Associates
P.O. Box 8413
Jupiter, FL 33468-8413

Dear Mr. Saporito:

On behalf of the U.S. Nuclear Regulatory Commission (NRC), I am responding to your petitions by letters dated June 26 and July 3, 2011 (Agencywide Documents Access and Management System (ADAMS) Accession Nos. ML11182B029 and ML11192A285, respectively), in which you requested escalated enforcement action against Fort Calhoun Station (FCS) and Cooper Nuclear Station (Cooper), respectively, regarding flood protection. Specifically, you asked the NRC to take action to suspend or revoke the NRC license(s) granted for the operation of these nuclear power reactors and issue a notice of violation with a proposed civil penalty against the collectively named and each singularly named licensee in the matters, in the amount of $500,000 for FCS and $1,000,000 for Cooper.

In your letter dated June 26, 2011, you also requested that the NRC issue a confirmatory order to Omaha Public Power District, the licensee for FCS, prohibiting the licensee from restarting FCS until such time as (1) the floodwaters subside to an appreciably lower level or to sea level, (2) the licensee upgrades its flood protection plan, (3) the licensee repairs and enhances its current flood protection berms, and (4) the licensee upgrades its station blackout procedures to meet a challenging extended loss-of-offsite power as a result of floodwaters and other natural disasters or terrorist attacks.

In your letter dated July 3, 2011, you requested that the NRC issue a confirmatory order to Nebraska Public Power District, the licensee for Cooper, requiring the licensee to bring Cooper to a cold-shutdown mode of operation until such time as (1) the floodwaters subside to an appreciably lower level or to sea level, (2) the licensee upgrades its flood protection plan, (3) the licensee repairs and enhances its current flood protection berms, and (4) the licensee upgrades its station blackout procedures to meet a challenging extended loss-of-offsite power as a result of floodwaters and other natural disasters or terrorist attacks.

As the basis of the request related to FCS, you stated the following:

> On June 26, 2011, a 2,000-foot berm at the Ft. Calhoun Nuclear Plant collapsed from the forces of flood waters surrounding the nuclear plant. The berm was constructed 16-feet wide at the base and 8-feet tall to provide flood protection for the nuclear plant's power-block. The licensee transferred the nuclear plant's off-site power to on-site diesel generators because of water leaking around the concrete berm surrounding the main transformers. In addition, flood-waters also surrounded auxiliary and containment buildings - designed to handle water up to 1,014-feet above sea level. NRC officials issued a statement to the media that -

> the licensee has an earthen berm to protect the electrical switch-yard and a concrete barrier surrounding electrical transformers.
>
> Petitioner contends here that (1) the licensee's installed flood-protection measures and systems and barriers at the Ft. Calhoun Nuclear Power Plant are not sufficient to adequately protect the nuclear reactor from a full-meltdown scenario like that currently unfolding in Japan; and (2) the licensee's station blackout procedures are not sufficient to meet a challenging extended loss of off-site power due to flood-waters and other natural disasters or terrorist attacks.

As the basis of the request related to Cooper, you stated the following:

> On June 19, 2011, the licensee notified the U.S. Nuclear Regulatory Commission (NRC) of an Unusual Event Declared at the Cooper Nuclear Station - in connection with the Missouri River flooding its banks. The NRC documented the licensee's notification as Event Number: 46969 indicating an Event Time of 04:02 CDT; and a Notification Time of 05:27 ET. During the context of the Unusual Event, the licensee maintain[ed] the nuclear reactor power at 100%. The licensee further communicated to the NRC that, *"The Missouri River is expected to crest at 899.5 feet within the next couple of days. It is expected that the elevation of the Missouri River will remain above 899 feet for most of the summer...."*
>
> Petitioner contends here that (1) the licensee's installed flood-protection measures and systems and barriers at the Cooper Nuclear Station are not sufficient to adequately protect the nuclear reactor from a full-meltdown scenario like that currently unfolding in Japan; (2) the licensee's station blackout procedures are not sufficient to meet a challenging extended loss of off-site power due to flood-waters and other natural disasters or terrorist attacks; (3) the licensee failed to timely notify the NRC of the Declaration of an Unusual Event within a one-hour period; and (4) the license continues to jeopardize public health and safety by failing to bring the Cooper Nuclear Station to a *"cold-shutdown"* mode of operation.

In accordance with Management Directive 8.11, "Review Process for 10 CFR 2.206 Petitions," dated October 25, 2000, the NRC has processed your letters under Title 10 of the *Code of Federal Regulations* (10 CFR) 2.206, "Requests for Action under this Subpart," and assigned these petitions to the NRC's Office of Nuclear Reactor Regulation.

On July 7 and 12, 2011, the NRC petition manager, Ms. Lynnea Wilkins, acknowledged receipt of your petitions, and you asked to address the Petition Review Board (PRB) before its meeting to make the initial recommendation to accept or reject your petitions for review. On August 29, 2011, you addressed the PRB during a teleconference to provide additional information in support of your petitions. During the teleconference, you asked the PRB to consider the information you provided as a supplement to your petitions. A copy of the transcript from the teleconference is available under ADAMS Accession No. ML11256A036.

On September 12 and October 13, 2011, the PRB met internally to discuss your petitions, as supplemented by the transcript. During both meetings, the PRB determined that it needed additional information from other internal resources before making its initial recommendation and a decision on the requests for immediate action. On November 28, 2011, the PRB again met internally to discuss your petitions. During this meeting, the PRB reached an initial recommendation that your petitions meet the criteria for review. The PRB also determined that there is no immediate safety concern that would warrant an immediate action by the NRC to prevent the restart of FCS or to bring Cooper to cold shutdown, as you requested. Therefore, the PRB has denied your request for immediate action.

Additionally, the PRB identified that your petitions raise several issues that are currently undergoing NRC evaluation as part of the agency's Near-Term Task Force review of insights from the Fukushima Dai-ichi accident in Japan, as documented in "Recommendations for Enhancing Reactor Safety in the 21st Century," dated July 12, 2011 (ADAMS Accession No. ML112510264), and in the associated staff requirements memorandum for SECY-11-0137, "Prioritization of Recommended Actions to be Taken in Response to Fukushima Lessons Learned," dated December 15, 2011 (ADAMS Accession No. ML113490055).

The PRB intends to use the results of the above review to inform its final decision on whether to implement the actions requested in your petition. In an e-mail dated December 13, 2011, the petitioner manager conveyed the PRB's decision to deny your requests for immediate action and the PRB's initial recommendation to accept the petitions for review. The petition manager also offered you a second opportunity to address the PRB by teleconference; however, you declined this opportunity.

Because you did not request to address the PRB upon receipt of this initial recommendation, the PRB's initial recommendation to accept your petitions for review has become the PRB's final recommendation.

As required by 10 CFR 2.206, the NRC will act on your petitions within a reasonable time. The petition manager, Ms. Lynnea Wilkins, can be reached at (301) 415-1377.

I have enclosed for your information a copy of the notice that the NRC is filing with the Office of the Federal Register for publication. I have also enclosed for your information a copy of Management Directive 8.11 and the associated brochure NUREG/BR-0200, "Public Petition Process," Revision 5, issued February 2003, prepared by the NRC Office of Public Affairs.

Sincerely,

Eric J. Leeds, Director
Office of Nuclear Reactor Regulation

Enclosures:
1. *Federal Register* Notice
2. Management Directive 8.11
3. NUREG/BR-0200

cc: Listserv

I have enclosed for your information a copy of the notice that the NRC is filing with the Office of the Federal Register for publication. I have also enclosed for your information a copy of Management Directive 8.11 and the associated brochure NUREG/BR-0200, "Public Petition Process," Revision 5, issued February 2003, prepared by the NRC Office of Public Affairs.

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Eric J. Leeds, Director
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cc: Listserv

DISTRIBUTION: G20110506/LTR-11-0402/EDATS: SECY-2011-0402
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ESmith, NRR/DSS/SBPB
VGoal, NRR/DE/EEEB
JUribe, NRR/DE/EMCB
DHoang, NRR/DE/EMCB

ADAMS Accession Nos.: Package ML113530557; Incoming ML11182B029 and ML11192A285; Letter ML120030022; FR Notice ML120030027; NUREG/BR-0200 ML050900248 *via email

OFFICE	NRR/DORL/LPL4/PM	NRR/DORL/LPL4/LA	QTE	NRR/DPR
NAME	LWilkins	JBurkhardt	KKribbs*	TMensah*
DATE	1/4/12	1/4/12	1/5/12	1/5/12
OFFICE	RIV/DRS/OB/BC	NRR/DE/EICB/BC	RES/DD	OGC
NAME	MHaire	GWilson	BHolian (A)*	
DATE			1/5/12	
OFFICE	NRR/DORL/LPL4/BC	NRR/DORL/D	NRR/D	
NAME	MMarkley	MEvans	ELeeds	
DATE				

OFFICIAL RECORD COPY

ENCLOSURE 1

FEDERAL REGISTER NOTICE

[7590-01-P]

NUCLEAR REGULATORY COMMISSION

Docket No. 50-285, License No. DPR-40; Docket No. 50-298, License No. DPR-46;

NRC-2012-XXXX

Request for Action against Omaha Public Power District and

Nebraska Public Power District

Notice is hereby given that by petitions dated June 26 and July 3, 2011, respectively, Thomas Saporito (the petitioner) has requested that the U.S. Nuclear Regulatory Commission (NRC or the Commission) take escalated enforcement actions against Omaha Public Power District, the licensee for Fort Calhoun Station, Unit 1 (FCS), and Nebraska Public Power District, the licensee for Cooper Nuclear Station (Cooper). The petitions dated June 26 and July 3, 2011, are publicly available in the NRC's Agencywide Documents Access and Management System (ADAMS) under Accession Nos. ML11182B029 and ML11192A285, respectively.

The petitioner has requested that NRC take action to suspend or revoke the NRC license(s) granted for the operation of nuclear power reactors and issue a notice of violation with a proposed civil penalty against the collectively named and each singularly named licensee in this matter – in the amount of $500,000 for Fort Calhoun Station and $1,000,000 for Cooper. Additionally, the petitioner requested that the NRC issue confirmatory orders to prohibit restart at FCS and to bring Cooper to a "cold shutdown" mode of operation until such time as (1) the flood-waters subside to an appreciable lower level or sea-level; (2) the licensee upgrades its flood-protection plan; (3) the licensee repairs and enhances its current flood-protection berms; and (4) the licensee upgrades its station blackout procedures to meet a challenging extended loss of off-site power due to flood-waters and other natural disasters or terrorist attacks.

As the basis for these requests, the petitioner stated that (1) the licensees' installed flood-protection measures and systems and barriers at FCS and Cooper are not sufficient to adequately protect the nuclear reactor from a full-meltdown scenario like that currently unfolding in Japan; and (2) the licensees' station blackout procedures are not sufficient to meet a challenging extended loss of off-site power due to flood-waters and other natural disasters or terrorist attacks.

The requests are being treated pursuant to Title 10 of the *Code of Federal Regulations* Section 2.206 of the Commission's regulations. The requests have been referred to the Director of the Office of Nuclear Reactor Regulation. As provided by Section 2.206, appropriate action will be taken on these petitions within a reasonable time. The petitioner requested an opportunity to address the Petition Review Board (PRB). The PRB held a recorded teleconference with the petitioner on August 29, 2011, during which the petitioner supplemented and clarified the petitions. The results of those discussions were considered in the PRB's determination regarding the petitioner's requests. As a result, the PRB acknowledged the petitioner's concerns regarding flood protection, including station blackout procedures, at FCS and Cooper. By letter dated January   , 2012 (ADAMS Accession No. ML120030022), the Director of the NRC's Office of Nuclear Reactor Regulation denied the petitioner's requests for immediate action. Additionally, the PRB noted that (1) natural disasters such as earthquakes and flooding and (2) station blackout regulations are undergoing NRC review as part of the lessons-learned from the Fukushima event. The PRB intends to use the results of the Fukushima review to inform its final decision on whether to implement the requested actions.

Copies of the petitions dated June 26 and July 3, 2011, are available for inspection at the NRC's Public Document Room (PDR), located at One White Flint North, Public File Area O1 F21, 11555 Rockville Pike (first floor), Rockville, Maryland 20852. Publicly available documents created or received at the NRC are accessible electronically through ADAMS in the NRC Library at http://www.nrc.gov/reading-rm/adams.html. Persons who do not have access to ADAMS or who encounter problems in accessing the documents located in ADAMS should contact the NRC's PDR Reference staff by telephone at 1-800-397-4209 or 301-415-4737, or by e-mail to PDR.Resource@nrc.gov.

Dated at Rockville, Maryland, this _____ day of January 2012.

FOR THE NUCLEAR REGULATORY COMMISSION.

Eric J. Leeds, Director,
Office of Nuclear Reactor Regulation.

ENCLOSURE 2

MANAGEMENT DIRECTIVE 8.11

ENCLOSURE 3

NUREG/BR-0200

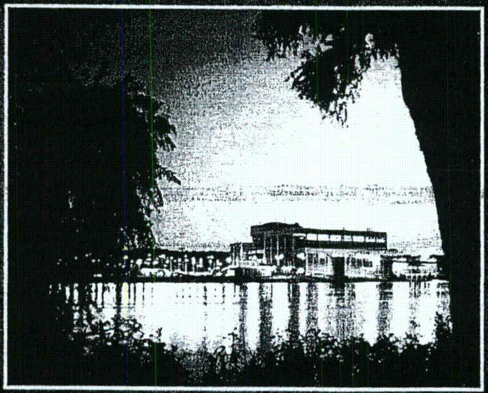
OPPD
Omaha Public Power District

Refer to Licensee in Entirety

Fort Calhoun Station

Flood Recovery Action Plan 4.1

Plant and Facility Geotechnical and Structural Assessment



HDR

December 28, 2011
Revision 2

Omaha Public Power District

Fort Calhoun Station

Flood Recovery Action Plan 4.1

Plant and Facility Geotechnical and Structural Assessment

December 28, 2011

Revision 2

Prepared for:
Omaha Public Power District
Fort Calhoun Station
9610 Power Lane
Blair, NE 68008

Prepared by:
HDR Engineering, Inc.
8404 Indian Hills Drive
Omaha, NE 68114

Professional Engineer Seal

[To be added.]

EXECUTIVE SUMMARY

Introduction

Omaha Public Power District's (OPPD's) Fort Calhoun Station (FCS) is a 484-megawatt nuclear power plant (OPPD, September 25, 2011). FCS is located on the west bank of the Missouri River in northeastern Washington County, Nebraska. FCS is approximately 4 miles southeast of Blair, Nebraska, and approximately 19 miles north of Omaha, Nebraska.

The flooding of the Missouri River during the summer of 2011 has "significantly challenged" the operation of FCS (OPPD, August 10, 2011). In response to this event, OPPD prepared a Flooding Recovery Action Plan that documented the actions necessary for the repair and restoration of FCS operations. This Fort Calhoun Station Plant and Facility Geotechnical and Structural Assessment Report (Assessment Report) has been prepared in response to FCS Flooding Recovery Action Plan 4.1, Plant and Facility Geotechnical and Structural Assessment.

Scope and Purpose

The FCS Plant and Facility Geotechnical and Structural Assessment has been completed to identify and describe the effects of the 2011 flood on 28 Priority 1 Structures and 19 Priority 2 structures at the site. Specifically, the objective of this Assessment Report is to present HDR's assessment of changes to the soil or rock that supports the structures at FCS that may have negatively impacted those structures.

Revision 0 of this Assessment Report was submitted to OPPD on October 14, 2011. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure. These assessments were incomplete in Revision 0 because the forensic investigation and/or monitoring for most of the Priority 1 Structures was not completed by the submittal date.

Revision 1 of this Assessment Report was submitted to OPPD on November 28, 2011. Revision 1 presented the partial and preliminary results of additional forensic investigation and monitoring to date for the Key Distress Indicators and the draft final assessment results for Priority 1 Structures.

Revision 2 of this Assessment Report presents the following:

- Complete and final results of the Key Distress Indicators forensic investigations
- Final assessment results for the Priority 1 Structures
- Final assessment results for the Priority 2 Structures
- Final results of the Comparative Geotechnical Analysis

Principal Findings of the Comparative Geotechnical Analysis

Comparison of geotechnical data for pre-flood and current investigations indicates that there was no observable difference in the overall geotechnical conditions at the site and that the foundation materials have not been disturbed or significantly weakened by the prolonged inundation caused by the 2011 flood. Comparison of seismic refraction data from the pre-flood and current investigations reveals similar magnitude of seismic wave velocities over the full depth of the overburden soils, and no observable differences between pre- and post-flood conditions were identified from this work.

Based on these findings and evaluations, the overall geotechnical conditions at the site have not been significantly altered due to the sustained high water. The observed scatter of data points is consistent with the relatively wide range of strength and stiffness and corresponding blow counts typically encountered in the alluvial soils within the Missouri River valley. However, these findings are considered applicable only to those soils present below a depth of 10 feet at the site. The upper 10 feet were hydro excavated to avoid damaging buried utilities. This upper layer may have been disturbed from underseepage beneath the temporary levees or from the settlement of utility backfill during drawdown of the river level and groundwater.

Detailed Forensic Investigations at Key Distress Indicators

Each structure was systematically observed for obvious signs of structural damage or distress caused by the 2011 flood. These inspections revealed three significant indicators of distress:

1. Increased groundwater flow into the Turbine Building sump
2. Pavement failure and sinkhole development in the paved access area between the Intake Structure and Service Building
3. Column settlement in the Maintenance Shop

Since publication of Revision 0, work has been ongoing to investigate subsurface conditions at each of the three Key Distress Indicators, as discussed below:

In the basement of the Turbine Building, 26 one-inch-diameter test holes were drilled through the floor slab to reach the subgrade. This work found that the Triggering Mechanism of subsurface piping of soil material due to the sump operation and seepage/flow into the drainage system pipes is occurring, and that the voids are significant and interconnected. Although it was also found that the foundation subgrade was not affected uniformly by the Triggering Mechanism, subsurface erosion/piping of soil from beneath the Turbine Building basement and perhaps beyond will continue as long as the drain system piping remains unrepaired. Voids, soft zones, and associated groundwater and piping flow paths will continue to enlarge and extend out from the drainage and sump system over time unless the flow of water into the sump system is stopped.

In the Paved Access Area, 40 one-inch-diameter test holes were drilled through the concrete paving slab, and six continuous SPT borings were completed. This work found no evidence of piping erosion, voids, or subsidence of site fills. Field testing of the subgrade exposed after concrete panel removal indicated that stiff to very stiff soils were generally encountered in the upper 3 feet below the ground surface or pavement. Based on the observations made and tests results obtained, the fill soils in the locations exposed and tested are compact, cohesive soils that are not susceptible to piping erosion. SPT borings did not identify voids or very soft/very loose conditions that might indicate piping or related material loss nor did they identify changes in soil relative density following the 2011 flood. Inclinometer and survey monitoring in the Paved Access Area indicates that movement of on-site subsurface soils or structures has not occurred.

In the Maintenance Shop, 16 one-inch-diameter test holes were drilled, followed by a second set of 6 one-inch-diameter test holes to investigate the settled column. The results of the KDI #3 forensic investigations have found that the distress observed in both the Maintenance Shop (failed column) and the Technical Support (cracked walls) are not associated with the Triggering Mechanism 7 - Soil Collapse (due to first time wetting). Therefore the CPFMs associated with this Triggering Mechanism (7a-7c) have been ruled out by this forensic investigation. The results show that the distress in both the Maintenance Shop and the Technical Support Center are connected to KDI #1, which is associated with the uncontrolled drainage of the groundwater into the broken Turbine Building basement drainage system piping. KDI #1 is associated with the Triggering Mechanism of Subsurface Erosion/Piping (due to pumping) and the CPFM applicable to the Maintenance Shop and Technical Support Center is 3a - Undermining and settlement of shallow foundation/slab (due to pumping). This CPFM will only be ruled out when the physical modifications presented for KDI #1, as presented in Section 4.1 of this Assessment Report, are implemented.

Recommendations

Turbine Building Sump

- OPPD should perform remedial work to stop the uncontrolled drainage of the groundwater into the broken Turbine Building basement drainage system piping and fill the voids beneath the basement floor slab.
- In addition to drainage system repair, the voids created by the subsurface erosion/ piping should be filled.
- A grouting program should be implemented to fill the voids and determine the volume of the voids.

Paved Access Area

- OPPD should complete their pavement restoration work.

Maintenance Shop

- Physical modifications to remediate the distress in the Maintenance Shop should be implemented as planned.
- Further investigations could be undertaken by OPPD as part of the design for the remedial work to repair the Maintenance Shop and Technical Support Center distress.
- Physical modifications outlined in the KDI #1 forensic investigations should be competed before the physical modifications to remediate the distress in the Maintenance Shop and Technical Support Center are implemented.

Priority 1 and Priority 2 Structural Assessments

The Geotechnical and Structural Assessments have been completed for each of the Priority 1 and Priority 2 Structures. In general, it has been determined that the 2011 Missouri River flood did not impact the geotechnical and structural integrity of the structures. However, in addition to the recommendations associated with the KDI investigations as described above, there are specific recommendations for remediation of 2011 flood impacts for seven structures as presented in each of their respective Section 5 and Section 6 assessments. Therefore, this determination is conditional upon implementation of those specific recommendations for the structures listed below:

Table E-1 - Priority 1 and Priority 2 Structures Having Specific Remediation Recommendations

Priority 1 Structures	Priority 2 Structures
Auxiliary Building	Service Building
Containment	Maintenance Shop
Technical Support Center	PA Paving, PA Sidewalks, and Outdoor Drives
Turbine Building	Potable Water Piping
Security Barricaded Ballistic Resistant Enclosures (BBREs)	Sanitary Sewer System

Table E-1 - Priority 1 and Priority 2 Structures Having Specific Remediation Recommendations	
Priority 1 Structures	**Priority 2 Structures**
Turbine Building South Switchyard	Shooting Range
Condensate Storage Tank	
Circulating Water System	
Raw Water Piping	
Fire Protection System Piping	
Waste Disposal Piping	
Fuel Oil Storage Tanks and Piping	
Main Underground Cable Bank, Auxiliary Building to Intake Structure	
Main Underground Cable Bank, MH-1 to Auxiliary Building	
Blair Water System	
Demineralized Water System	

HDR has concluded that the 2011 Missouri River flood did not impact the geotechnical and structural integrity of the following structures because the potential for failure of this structure due to the flood is not significant.

Table E-2 - Priority 1 and Priority 2 Structures Not Impacted by 2011 Flood	
Priority 1 Structures	**Priority 2 Structures**
Intake Structure	New Warehouse
Rad Waste Building	Chemistry/Radiation Protection (CARP) Building
Independent Spent Fuel Storage Installation (ISFSI)	Maintenance Fabrication Shop
Security Building	Maintenance Storage Building
Underground Cable Trench (Trenwa)	Old Warehouse
Demineralized Water Tank, Pump House, and Reverse Osmosis (RO) Unit	Training Center
Meteorological Tower and Miscellaneous Structures	Administration Building
Original Steam Generator Storage Building (OSGS)	Hazardous Material Storage Building
Switchyard	Maintenance Garage
Transmission Towers	Tertiary Building (Boat Storage)
River Bank	Spare Transformer Pads
Camera Towers and High Mast Lighting	Gravel Parking Lots
	Sewage Lagoons

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Acronyms and Abbreviations

ANSS	Advanced National Seismic System
BBRE	Barricaded Ballistic Resistant Enclosure
CARP	Chemistry/Radiation Protection Building
CEUS	Central and Eastern United States
cfs	cubic feet per second
CHDPE	corrugated high density polyethylene
CMP	corrugated metal pipe
CMU	concrete masonry unit
CPFM	credible PFM
CPT	cone penetration test
CQE	Critical Quality Element
EAR	Engineering Assistance Request
el.	elevation
FAA	Federal Aviation Administration
FCS	Fort Calhoun Station
FMEA	Failure Modes and Effects Analysis
fps	feet per second
FRP	fiberglass reinforced plastic
ft	feet
g	acceleration due to gravity
gpm	gallon per minute
GPR	ground-penetrating radar
HD	high density
HDR	HDR Engineering, Inc.

HVAC	heating, ventilation, and air conditioning
in.	inches
ICF	insulated concrete forms
ISFSI	Independent Spent Fuel Storage Installation
ISO	International Organization for Standardization
kV	kilovolt
LOCA	loss of coolant accident
MAF	million acre-feet
MCE	maximum credible earthquake
Met	Meterological
MH	Manhole
MSL	mean sea level
N value	blows per foot
NAVD 88	North American Vertical Datum of 1988
NEI	Nuclear Energy Institute
NGVD 29	National Geodetic Vertical Datum of 1929
NOUE	Notification of Unusual Event
NQA-1	Nuclear Quality Assurance
OPPD	Omaha Public Power District
OSGS	Original Steam Generator Storage Building
P&ID	piping and instrumentation diagrams
PA	Protected Area
PBD	Program Basis Document
PFM	potential failure mode
PGA	peak ground acceleration
PSF	pounds per square foot

psi	pounds per square inch
PVC	polyvinyl chloride
QA	Quality Assurance
QC	Quality Control
QCP	Quality Control Plan
RCP	reinforced concrete pipe
RM	River Mile
RQD	rock quality designation
SP	poorly graded sand
SPT	standard penetration test
U.S.	United States
USACE	U.S. Army Corps of Engineers
USDA-NRCS	U.S. Department of Agriculture, Natural Resource Conservation Service
USGS	U.S. Geolocial Survey
VCP	vitrified clay pipe

Definitions

Class I	Class I indicates a system, structure, or component, including instruments and controls, whose failure might cause or increase the severity of an accident that could result in an uncontrolled release of radioactivity. This classification also includes components and structures vital to safe shutdown and isolation of the reactor.
Confidence	Confidence is an opinion regarding the need for additional information.
Credible PFMs (CPFMs)	CPFMs were those that were significant enough to demand further investigation and evaluation or studies that would increase the confidence in the findings or change the conclusion.
Critical Quality Element (CQE)	CQEs are structures, systems, components, or items whose satisfactory performance is required to prevent or mitigate the consequences of postulated accidents that could cause undue risk to the health and safety of the public.
Degradation	Degradation is a negative change to the soil or rock that supports a structure, caused by the sustained inundation of the FCS site during the summer of 2011, which could materially and negatively impact the integrity or intended function of the structure.
Key Distress Indicator	A Key Distress Indicator is an observed problem area that potentially indicated that the 2011 flood had changed the site's geotechnical and physical character.
Non-credible PFMs	Non-credible PFMs are those that were clearly so remote that they were considered negligible risk contributors.
Potential Failure Mode (PFM)	PFMs are the ways in which a structure might fail. Failures are any errors or defects, and can be potential or actual.
Potential for degradation/direct floodwater impact	a determination of whether the Triggering Mechanisms for the CPFMs could have been or were actually initiated by the flood.
Priority 1 Structures	Priority 1 Structures are those structures and systems that directly support plant operations.
Priority 2 Structures	Priority 2 Structures are those structures and systems that do not directly support plant operations.
Significance	Significance is determined by the combined consideration of two elements. The first element is the potential for degradation as described above. The second is the implications of that degradation to a structure built to its specific design standard.

Triggering Mechanism	Triggering Mechanisms are flood-induced triggering mechanisms that could have caused degradation of the soil and/or rock that supports the FCS structures and/or could have caused direct impacts on structures due to the force of the floodwater. Triggering Mechanisms could lead to a potential failure mode (PFM).

Report Contributors

Name	Project Role	Education and Experience
Project Management		
David Rohan, PE	Project Manager	M.S. Industrial/Manufacturing Engineering B.S. Mechanical Engineering 17 years of experience
Lawrence Cieslik, PE	Project Principal	B.S. Civil Engineering 36 years of experience
Tom Sanders, PE	Project Principal	M.S. Civil Engineering B.S. Civil Engineering 33 years of experience
Michael Siedschlag, PE	Project Principal	B.S. Civil Engineering 37 years of experience
Barry Butterfield	Quality Assurance/Quality Control	M.S. Civil Engineering B.S. Civil Engineering 37 years of experience
Craig Osborn, PE	Quality Assurance/Quality Control	B.S. Civil Engineering B.S. Conservation/Renewable Natural Resources 36 years of experience
Baseline Condition		
Michael Butterfield, PE	Civil Engineer	B.S. Civil Engineering 9 years of experience
John Charlton	Engineering Geologist	M.S. Geological and Related Sciences M.B.A. Business Administration B.A. History 19 years of experience
Charles Hookham, PE	Structural Engineer	M.B.A. Business Administration B.S. Civil Engineering 31 years experience
Andrew McCoy, PhD, PE	Water Resources Engineer	Ph.D. Civil Engineering M.S. Civil Engineering B.S. Civil Engineering 12 years of experience
Christopher Miller, PE	Structural Engineer	M.S. Structural Engineering B.S. Civil Engineering B.Arch. Architecture 33 years of experience
Elena Sossenkina, PE	National Technical Advisor for Dams, Levees, and Hydraulic Structures	M.S. Mechanical Engineering 15 years of experience

Name	Project Role	Education and Experience
Civil Engineering		
Richard Niedergeses, PE	Civil Engineering Team Lead	B.S. Civil Engineering 39 years of experience
Anna Grimes, PE	Civil Engineer	B.C.E. Civil Engineering 26 years of experience
Brian Hindley, PE	Civil Engineer	B.S. Civil Engineering 6 years of experience
Richard Madson, PE	Civil Engineer	B.S. Civil Engineering 23 years of experience
Hugh O'Grady, PE	Civil Engineer	B.E. Civil Engineering 26 years of experience
John Smith	Civil Engineer	B.S. Civil Engineering 5 years of experience
Geotechnical Engineering		
John Christiansen, PE	Geotechnical Engineering Team Lead	B.S. Civil Engineering 23 years of experience
Justin Anderson	Geotechnical Engineer	M.S. Civil Engineering B.S. Civil Engineering 5 years of experience
Rolland Boehm, PE	Senior Geotechnical Engineer	M.S. Geotechnical Engineering B.S. Civil Engineering 24 years of experience
Bryan Kumm, PE	Geotechnical Engineer	B.C.E. Civil Engineering 6 years of experience
Gregg Mitchell, PG	Engineering Geologist	B.S. Environmental Technology A.A. Liberal Arts/Sciences 23 years of experience
Steve Olson, PE	Senior Geotechnical Engineer	B.C.E. Civil Engineering 28 years of experience
Patrick Poepsel, PE	Geotechnical Engineer	M.E. Geotechnical Engineering M.S. Geotechnical Engineering B.S. Civil Engineering 27 years of experience
Structural Engineering		
Keith Kirchner, PE	Structural Engineering Team Lead	M.S. Civil Engineering B.S. Civil Engineering 32 years of experience
Cameron Collingsworth	Structural Engineer	M.S. Architectural Engineering B.S. Architectural Engineering 3 years of Experience
Keith Froscheiser, PE	Structural Engineer	B.S. Civil Engineering 17 years of experience

Name	Project Role	Education and Experience
Nick Lampe, PE	Structural Engineer	M.S. Civil Engineering B.S. Civil Engineering 13 years of experience
Engineering Support		
Joshua Miller	Survey Monitor	B.S. Mechanical Engineering 7 years of experience
Senior Review Team		
Keith Ferguson, PE	National Practice Leader for Dams, Levees, and Hydraulic Structures	M.S. Civil Engineering B.S. Civil Engineering 33 years of experience
Les Harder, PhD, PE	Senior Water Resources Technical Advisor	Ph.D. Civil Engineering M.S. Civil Engineering B.S. Civil Engineering 36 years of experience
Christopher Miller, PE	Senior Structural Engineer	M.S. Structural Engineering B.S. Civil Engineering B.Arch. Architecture 33 years of experience
Document Support		
Louise Baxter	Technical Editor	B.S. Political Science 10 years of experience
Kimberly Gust	Technical Editor	M.A. English Composition and Rhetoric B.S.E. English 14 years of experience
Klayton Kasperbauer	Copy Editor	B.A. English (expected May 2012) Less than 1 year of experience
Amy Sorensen	GIS Analyst	A.S. Geographic Information Systems A.S. Education 7 years of experience

SECTION 1.0
INTRODUCTION

1.0 INTRODUCTION

Omaha Public Power District's (OPPD's) Fort Calhoun Station (FCS) is a 484-megawatt nuclear power plant (OPPD, September 25, 2011). FCS is located on the west bank of the Missouri River in northeastern Washington County, Nebraska. FCS is located approximately 4 miles southeast of Blair, Nebraska, and approximately 19 miles north of Omaha, Nebraska.

Massive flooding in the Missouri River basin occurred in 2011, as described in Section 1.3, Background. Because FCS is located along the Missouri River, floodwater encroached on the FCS site. In June 2011, OPPD contracted HDR Engineering, Inc. (HDR) to provide professional engineering services in support of OPPD's Fort Calhoun Station Flooding Recovery Action Plan. HDR provided specialized engineering services for the assessment of geotechnical and structural changes caused by the 2011 Missouri River flood.

The flooding of the Missouri River during the summer of 2011 has "significantly challenged" the operation of FCS (OPPD, August 10, 2011). In response to this event, OPPD prepared a Flooding Recovery Action Plan that documented the actions necessary for the repair and restoration of FCS operations. This Fort Calhoun Station Plant and Facility Geotechnical and Structural Assessment Report (Assessment Report) has been prepared in response to FCS Flooding Recovery Action Plan 4.1, Plant and Facility Geotechnical and Structural Assessment.

1.1 Scope and Purpose

The FCS Plant and Facility Geotechnical and Structural Assessment has been completed to identify and describe the geotechnical and structural effects of the 2011 flood on 28 Priority 1 Structures and 19 Priority 2 Structures at the site. The Priority 1 Structures are those structures and systems that directly support plant operations. These structures are listed in Table 1-1. Priority 2 Structures are those structures and systems that do not directly support plant operations. These structures are listed in Table 1-2. Specifically, the purpose of this Assessment Report is to present HDR's assessment of changes to the soil or rock that supports the structures at FCS due to the 2011 Missouri River flood and/or the direct impacts of floodwater that may have negatively impacted those structures.

Table 1-1 – Priority 1 Structures (Must Be Assessed Prior to Plant Restart)	
Class I (Seismic) Structures	**Non-Class I Structures Outside Protected Area**
Intake Structure	Original Steam Generator Storage Building (OSGS)
Auxiliary Building	Switchyard
Containment	Transmission Towers
Rad Waste Building	Meteorological Tower
Technical Support Center	Demineralized Water Tank, Pump House, and Reverse Osmosis (RO) Unit
Non-Class I Structures Inside Protected Area	Underground Utilities
Independent Spent Fuel Storage Installation (ISFSI)	Blair Water System
Security Building	Main Underground Cable Bank, MH-1 to Auxiliary Building (MH-1, MH-2, MH-3, MH-4)
Turbine Building	
Security Barricaded Ballistic Resistant Enclosures (BBREs)	River Bank
Turbine Building South Switchyard	
Condensate Storage Tank	
Underground Utilities	
Underground Cable Trench (Security Trenwa)	
Circulating Water System	
Demineralized Water System	
Raw Water Piping	
Fire Protection System Piping	
Waste Disposal Piping	
Fuel Oil Storage Tanks and Piping (only FO-1, FO-10, and FP-1B)	
Main Underground Cable Bank, Auxiliary Building to Intake Structure (Manhole [MH]-5, MH-31)	
Camera Towers and High Mast Lighting	
Source: OPPD. August 10, 2011. *Flooding Recovery Action Plan, Revision 0.* Document number LIC-11-0090.	

Table 1-2 – Priority 2 Structures (Do Not Directly Support Plant Operations)	
Non-Class I Structures Inside Protected Area	**Non-Class I Structures Outside Protected Area**
New Warehouse	Maintenance Storage Building (Maintenance Shed)
Service Building	Old Warehouse
Chemistry/Radiation Protection (CARP) Building	Training Center
Maintenance Shop	Administrative Building
Maintenance Fabrication Shop	Hazardous Material Storage Building (Hazmat Shed)
Protected Area Paving and Sidewalks	Maintenance Garage
Underground Utilities	Tertiary Building (Boat Storage)
Potable Water	Spare Transformer Pads
Sanitary Sewer	Shooting Range
	Gravel Parking Lots
	Outdoor Concrete Slabs and Driveways
	Underground Utilities
	Potable Water
	Sanitary Sewer
	Sewage Lagoons
Source: OPPD. August 10, 2011. *Flooding Recovery Action Plan, Revision 0*. Document number LIC-11-0090.	

1.2 Assessment Report Organization, Content, and Revision History

1.2.1 Document Organization

This Assessment Report is organized as follows:

- Section 1.0, Introduction
- Section 2.0, Site History, Description, and Baseline Condition
- Section 3.0, Assessment Process, Procedures, and Methods
- Section 4.0, Key Distress Indicators
- Section 5.0, Priority 1 Structures
- Section 6.0, Priority 2 Structures (to be included in a future revision of the Assessment Report)
- Section 7.0, Summary and Conclusions
- Section 8.0, References
- Section 9.0, Attachments

1.2.2 Document Content

This report presents the findings, conclusions, and recommendations for the geotechnical, structural, and civil aspects of HDR's inspection completed at the FCS site. It has been prepared in accordance with generally accepted engineering practice and in a manner consistent with the level of care and skill required for this type of project within this geographical area. No warranty, expressed or implied, is made.

The findings, conclusions, and recommendations presented herein are based on systematic and thorough visual observations and reconnaissance, review of available design and construction information provided by others, the results of field exploration and laboratory materials testing, the results of engineering evaluations, and HDR's experience and engineering judgment.

Geotechnical engineering and the geologic sciences are characterized by uncertainty. Professional judgments presented herein are based partly on HDR's understanding of the past construction at the FCS site, information gathered during the inspection, HDR's general experience, and the state of the practice at the time of this writing.

For structures that were or potentially could still be impacted by the 2011 inundation of the FCS site, this Assessment Report presents recommendations for 1) additional detailed forensic investigations, 2) additional monitoring, or 3) physical modifications. This Assessment Report is not intended to modify the accepted design basis of each structure, or to modify any accepted emergency action plan for FCS.

1.2.3 Revision History

Revision 0 of this Assessment Report was submitted to OPPD on October 14, 2011. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure. Since then, additional surveys and site monitoring activities were conducted that have changed the assessment results for some structures, and are included in this Revision 1. Table 1-3 summarizes the revision history of this document.

Table 1-3 – Assessment Report Revision History

Revision Number	Date of Issuance	Changes
0	October 14, 2011	NA
1	November 28, 2011	Incorporates results of the following: • Geotechnical summary, including the majority of the data from subconsultants • Geotechnical comparative analysis • Additional site monitoring
2 (Draft)	December 28, 2011	Incorporates results of the following: • Forensics investigations for Key Distress Indicators • Assessment of Priority 2 Structures

1.3 Background

FCS shut down on April 9, 2011, for a scheduled maintenance and refueling outage. The refueling and maintenance activities proceeded until a combination of above-normal snowpack in the plains in the Northern United States (U.S.), above-normal snowpack in the mountains above Fort Peck Dam[1] on the Missouri River, and excessive upstream spring rains in eastern Montana and North and South Dakota resulted in massive flooding in the Missouri River basin. The U.S. Army Corps of Engineers (USACE) began releasing record discharges from Gavins Point Dam[2] in late May 2011. The hydrologic background of this Missouri River flood event is explained in Section 2.3 of this Assessment Report.

1 Fort Peck Dam is the uppermost in a series of six mainstem dams on the Missouri River.
2 Gavins Point Dam is the lowermost of six mainstem dams on the Missouri River.

The plant was in cold shutdown when, on June 6, 2011, FCS entered Notification of Unusual Event (NOUE) status as floodwater on the site exceeded an elevation of 1004 feet (ft).[3] After this declaration, the Missouri River continued to rise as increasing amounts of water were released from upstream dams. Floodwater covered much of the FCS site, reaching a maximum elevation of approximately 1006.9 ft. The average elevation of the site surrounding the Containment, Turbine Building, and Auxiliary Building is approximately 1004 ft. A variety of steps were taken to prevent floodwater from entering any critical buildings on site. The measures taken to protect the Priority 1 and Priority 2 structures are listed in Tables 1-4 and 1-5, respectively.

Table 1-4 – Summary of Flood Protection Measures Taken for Priority 1 Structures

Priority 1 Structure	Method of Flood Protection
Intake Structure	Structural flood-proofing, walkway access
Auxiliary Building	Aqua Dam[A]
Containment	Aqua Dam
Rad Waste Building	Aqua Dam
Technical Support Center	Aqua Dam
Independent Spent Fuel Storage Installation (ISFSI)	Sandbag levee
Security Building	HESCO barrier[B]
Turbine Building	Aqua Dam
Security Barricaded Ballistic Resistant Enclosures (BBREs)	Varies (see below)
BBRE F-1	Aqua Dam
BBRE F-2	Aqua Dam
BBRE F-3	None (walkway access)
BBRE F-4	None (walkway access)
BBRE F-5	None (walkway access)
BBRE F-6	None (walkway access)
Turbine Building South Switchyard	Aqua Dam
Condensate Storage Tank	None
Demineralized Water Tank and Pumphouse	Aqua Dam
Meteorological (Met) Tower and Miscellaneous Structures	None
Original Steam Generator Storage Building (OSGS)	None
Switchyard	Temporary earthen berm/sandbag levee
Transmission Towers	None

[A] - An Aqua Dam is an engineered water barrier used to contain, divert, and control the flow of water. It consists of two polyethylene liners contained by a single woven geo-tech outer tube. When the two inner tubes are filled with water, the resulting pressure and mass create a stable, non-rolling wall of water (Layfield Environmental Systems, 2008).

[B] - A HESCO barrier is a collapsible container used to block and control floodwater and debris. Composed of wire-mesh with heavy-duty polypropylene geotextile liner, HESCO barriers are filled with aggregate and placed as temporary dikes or flood defense walls.

[3] All elevations are expressed in National Geodetic Vertical Datum of 1929 (NGVD 29), also known as the Sea Level Datum of 1929.

Table 1-5 – Summary of Flood Protection Measures Taken for Priority 2 Structures	
Priority 2 Structure	**Method of Flood Protection**
New Warehouse	None
Service Building	Aqua Dam[A]
Chemistry/Radiation Protection (CARP) Building	Aqua Dam
Maintenance Shop	Aqua Dam
Maintenance Fabrication Shop	None
Maintenance Storage Building (Maintenance Shed)	None
Old Warehouse	Aqua Dam around northern portion of building
Training Center	Aqua Dam
Administrative Building	Aqua Dam
Hazardous Material Storage Building (Hazmat Shed)	Unknown
Maintenance Garage	None
Tertiary Building (Boat Storage)	None
Spare Transformer Pads	Varies (see below)
T1 Spare Transformer Pad	Earthen berm covered with crushed rock
Spare pad located west of the T1 Spare Transformer Pad	Sandbag levee
Shooting Range	Berm
[A] - An Aqua Dam is an engineered water barrier used to contain, divert, and control the flow of water. It consists of two polyethylene liners contained by a single woven geo-tech outer tube. When the two inner tubes are filled with water, the resulting pressure and mass create a stable, non-rolling wall of water (Layfield Environmental Systems, 2008).	

The peak release from Gavins Point Dam was 160,000 cubic feet per second (cfs), which was reached on June 26, 2011, and releases remained at that level until mid-August. USACE's forecast on November 1, 2011, estimated that in 2011, runoff into the Missouri River above Sioux City would be nearly 61 million acre-feet (MAF). This is the highest amount of runoff since 1898, eclipsing the previous high runoff of 49 MAF. Beginning on August 19, 2011, USACE began reducing releases daily in 5,000 cfs increments, and water levels began to decline. FCS remained in emergency status until August 29, 2011, when floodwater fell below elevation (el.) 1004 ft. The site was in an emergency condition for 84 days.

Since then, OPPD has been actively engaged in cleaning up deposited sediment from the parking lots and roadways, removing flood debris, repairing obvious flood damage, and conducting the plant activities necessary to resume generation. Preparation of this Assessment Report is part of these activities.

1.4 Assessment Process

The post-flooding assessment of FCS structures was completed by first conducting a systematic and thorough visual observation of each structure to identify any outward signs of distress caused by the flood. After the visual observations, data on the 2011 flood, including the areal extent, water depths, water velocities, and the effect on groundwater at the FCS site, were compiled. Baseline data for the geology, geomorphology, geotechnical, and design conditions prior to the 2011 flood were also compiled. A list of flood-induced triggering mechanisms that could have caused degradation to the soil and/or rock that supports the FCS structures and/or could have caused direct impacts on structures due to the force of the floodwater (Triggering Mechanisms) was then developed. Examples of Triggering Mechanisms include settlement, erosion, stability, hydraulic actions, and frost actions. Using the list of potential Triggering Mechanisms, a comprehensive list of potential failure modes (PFMs) was developed. PFMs are the ways in which a structure might fail. Failures are any errors or defects, and can be potential or actual. Examples of PFMs include undermining and settlement of shallow foundation/slab, undermined buried utilities, and loss of lateral support for pile foundations. Using the knowledge compiled for the baseline on each structure's design standard (for example, shallow or deep founded building or buried utility), a list of corresponding PFMs was compiled for each structure from the comprehensive list of PFMs. A more detailed discussion of the assessment process is provided in Section 3.0 of this Assessment Report.

1.5 Quality Assurance and Control

HDR has developed a Quality Control Plan (QCP), which supplements HDR's Quality Assurance/Quality Control (QA/QC) Program Manual, to provide guidance for performing QA evaluations of assessment activities. HDR's program is based on International Organization for Standardization (ISO) 9000 principles. HDR's QA/QC process is not certified as a Nuclear Quality Assurance (NQA-1) program. The Project QCP ensures that QA and QC activities are documented and performed in accordance with written procedures or checklists.

SECTION 3.0
ASSESSMENT PROCESS, PROCEDURES, AND METHODS

3.0 ASSESSMENT PROCESS, PROCEDURES, AND METHODS

The purpose of the assessment process is to qualitatively determine the significance of the potential for failure of each FCS structure due to the effects of the 2011 Missouri River flood. This section of the Assessment Report presents detail on the steps in the assessment process, a description of the methods used during the field observations, a list of all of the potential failure modes (PFMs) that were identified, a list of the PFMs determined to be "non-credible" in the initial screening (prior to the detailed assessments), and the reasons for their elimination. This section also presents information on the assessment methods used to determine the significance of the potential for failure due to the 2011 Missouri River flood.

3.1 Assessment Process

As discussed in Section 1.1, the purpose of this Assessment Report is to present HDR's assessment of changes to the soil or rock that supports the structures at FCS due to the 2011 Missouri River flood and/or any direct impacts of floodwater that may have negatively impacted those structures. Structures to be assessed were selected and prioritized by OPPD (see Table 1-1) and include buildings, process structures, equipment foundations, tank foundations, and electrical towers, all of which are referred to as structures in this Assessment Report.

The post-flooding assessment of FCS structures was completed by first conducting a systematic and thorough visual observation of each structure to identify any outward signs of distress caused by the flood. After the visual observations, data on the 2011 flood, including the areal extent, water depths, water velocities, and the effect on groundwater at the FCS site, were compiled. Baseline data for the geology, geomorphology, geotechnical, and design conditions prior to the 2011 flood were also compiled. A list of flood-induced triggering mechanisms that could have caused degradation to the soil and/or rock that supports the FCS structures and/or could have caused direct impacts on structures due to the force of the floodwater (Triggering Mechanisms) was then developed. Examples of Triggering Mechanisms include settlement, erosion, stability, hydraulic actions, and frost actions. Using the list of potential Triggering Mechanisms, a comprehensive list of PFMs was developed. PFMs are the ways in which a structure might fail. Failures are any errors or defects, and can be potential or actual. Examples of PFMs include undermining and settlement of shallow foundation/slab, undermined buried utilities, and loss of lateral support for pile foundations. Using the knowledge compiled for the baseline on each structure's design standard (for example, shallow or deep founded building or buried utility), a list of corresponding PFMs was compiled for each structure from the comprehensive list of PFMs. A detailed list of Triggering Mechanisms and PFMs is presented in Section 3.4.

Once the list of PFMs was compiled for each structure, these PFMs were screened to determine if they were "credible" (CPFMs), which means a particular PFM could have occurred or could be in progress due to the changes caused by the 2011 flood. This included a determination of whether the Triggering Mechanisms for the CPFMs could have been or were actually initiated by the flood (potential for degradation/direct floodwater impact). As a result, some PFMs were determined to be non-credible. For example, PFMs arising from river bank erosion were eliminated because no evidence of bank erosion was observed. A detailed list of PFMs eliminated from detailed study is presented in Section 3.6.

During detailed assessment, when additional data were available including the results of the systematic visual observations, a secondary screening took place to rule out additional CPFMs. This might have resulted in the elimination of all of the CPFMs initially identified for a particular structure, or there could be remaining CPFMs, which are discussed in detail in this Assessment Report. Also, the PFMs screened out as non-credible in the initial screening described above were reviewed again in light of the additional available data to determine if they should be added back to the list of CPFMs. The remaining CPFMs were evaluated to determine first the potential for degradation to the soil or rock that supports the structure and/or the direct floodwater impacts due to the 2011 flood and then the implications of that degradation to a structure of that particular design type. The combination of the potential for degradation/direct floodwater impact and of the implications of that degradation/impact is termed the "potential for failure" and is then categorized as "significant" or "not significant." The final step in the analysis was to evaluate the "confidence" in the potential-for-failure determination as either "low" or "high."

3.2 Assessment Process Steps

The purpose of the assessment process is to qualitatively determine the significance of the potential for failure of each FCS structure due to the effects of the 2011 flood. The assessment process involved eight steps, as shown in Figure 3-1. In addition, the assessment process has several feedback loops to allow for incorporation of new information as it becomes available and revision of the subsequent steps as appropriate.

Step 1. **Site Description and Baseline Condition/History** – Review construction documents, as-built drawings, previous reports, and plant performance history to determine the pre-flood conditions at the site. This step is necessary to allow a comparison of the pre-flood and post-flood conditions. Baseline information for the FCS site and structures was compiled to include data on the geology, geomorphology, geotechnical, and design conditions prior to the 2011 flood. In addition, data on the 2011 flood itself, including the areal extent, water depths, water velocities, and the effect on groundwater at the FCS site were compiled. The baseline condition and history as it pertains to the various structures at the site is provided in Section 2.0 of this Assessment Report.

Step 2. **Potential Failure Modes** – Using the compiled data on the 2011 flood in Step 1, develop a list of Triggering Mechanisms. Using the list of potential Triggering Mechanisms, develop a comprehensive list of PFMs. Using the knowledge compiled for the baseline on each structure's design standard, select the corresponding PFMs for each structure from the comprehensive list. The list of identified PFMs is presented in Section 3.4.

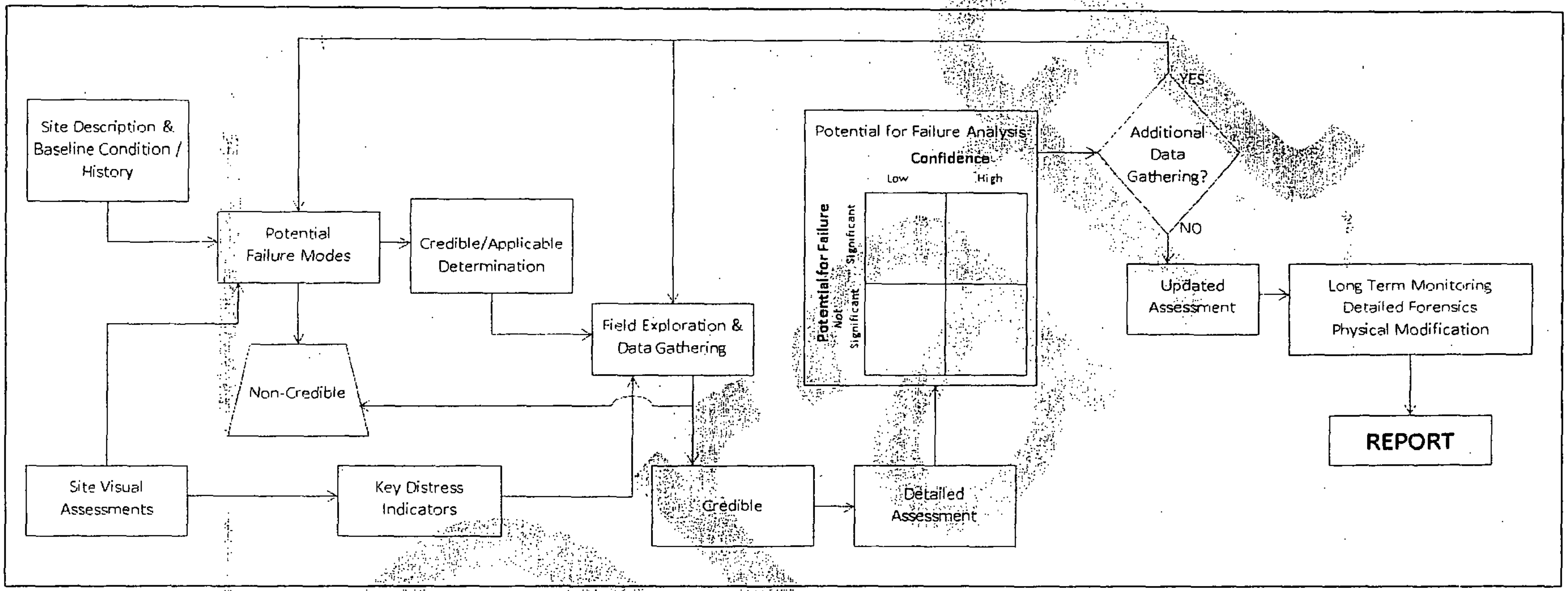


Figure 3-1 – Plant and Facility Geotechnical and Structural Assessment Process

Step 3. **Credible/Applicable Determination** – Conduct initial screening of Triggering Mechanisms and PFMs to determine if a specific PFM is applicable, credible, or non-credible for a particular structure. The initial screening is based on general review of background information, prior knowledge of the site, and observations from the initial site inspection(s). In this step, PFMs are categorized as one of following:

- Not Applicable – PFMs that are not applicable to that type of structure (For example, "loss of lateral support for pile foundation" would not apply to a structure that does not have a pile foundation.)
- Credible – PFMs that are 1) physically possible, and 2) significant enough to be further evaluated
- Non-credible – PFMs (or their associated Triggering Mechanisms) for which the chance of their existence is judged to be so small, based on the available information, that they are considered negligible contributors to the potential for failure

Develop methods and procedures for evaluating CPFMs, including the scope and objectives for various field exploration activities, distress indicators to look for in the field, and a list of baseline data required for the evaluation of a particular CPFM.

Step 4. **Field Exploration and Data Gathering** – Conduct field visits, geophysical and geotechnical testing, laboratory testing, structural condition assessment, civil inspections, field survey, and other field data gathering. This step also includes additional research of existing OPPD documents to identify basis of design, construction details, and performance history of a structure or system in question.

Step 5. **Credible** – Reassess each CPFM identified in Step 3 using the additional data and analysis to determine if any of the CPFMs should be "ruled out" prior to detailed assessment. In addition, review the PFMs screened out as non-credible in the initial screening described above in light of the additional available data to determine if they should be added back to the list of CPFMs. This could result in the elimination of all of the CPFMs initially identified for a particular structure, or there might be remaining CPFMs that will be carried forward for detailed assessment.

Step 6. **Detailed Assessment** – Conduct a detailed assessment of each remaining CPFM for each structure to identify changes from the baseline conditions. Determine whether the Triggering Mechanisms for the CPFMs were actually initiated by the flood (potential for degradation/direct floodwater impact).

Step 7. **Potential for Failure Analysis** – Given the potential for degradation/direct floodwater impact as identified in Step 6, determine the significance of the potential for failure. The significance of the potential for failure is determined by the combined consideration of two elements: the first element is the potential for degradation/direct floodwater impact, and the second is the implications of that degradation/direct floodwater impact to a structure built to its specific design standard.

The rationale for the potential-for-failure significance determination, including a description of the role each element played in that determination, is provided in Sections 5.0 and 6.0 of this Assessment Report for Priority 1 and Priority 2 Structures, respectively:

- **Not Significant/High Confidence** – "Not Significant" indicates that the potential for failure (the combined consideration of the potential for degradation/direct floodwater impact and the implications of that degradation/direct floodwater impact to a structure built to its specific design standard) has been qualitatively evaluated as "low." A description of the reason why a CPFM for any particular structure was placed in this category, including a description of the role each element played in the significance determination, is provided in Sections 5.0 and 6.0 of this Assessment Report for Priority 1 and Priority 2 Structures, respectively. "High Confidence" indicates that additional information and studies are not likely to increase the confidence in the findings or change the conclusions. By definition, all of the non-credible PFMs (see Tables 3-3 and 3-4) and "ruled out" CPFMs fall into this category. There are no recommended actions identified for any CPFMs listed in this category.
- **Not Significant/Low Confidence** – "Not Significant" indicates that the potential for failure (the combined consideration of the potential for degradation/direct floodwater impact and the implications of that degradation/direct floodwater impact to a structure built to its specific design standard) has been qualitatively evaluated as "low." A description of the reason why a CPFM for any particular structure was placed in this category, including a description of the role each element played in the significance determination, is provided in Sections 5.0 and 6.0 of this Assessment Report for Priority 1 and Priority 2 Structures, respectively. "Low Confidence" indicates that additional information and studies are required to increase confidence in the findings. The CPFMs included in this category are those for which additional data are required to confirm that there are "no further recommended actions."
- **Significant/Low Confidence** – "Significant" indicates that the potential for failure (the combined consideration of the potential for degradation/direct floodwater impact and the implications of that degradation/direct floodwater impact to a structure built to its specific design standard) has been qualitatively evaluated as "high." A description of the reason why a CPFM for any particular structure was placed in this category, including a description of the role each element played in the significance determination, is provided in Sections 5.0 and 6.0 of this Assessment Report for Priority 1 and Priority 2 Structures, respectively. "Low Confidence" indicates that additional information and studies are required to increase the confidence in the findings. The CPFMs included in this category are those for which additional data are required to determine whether physical modification will be recommended.
- **Significant/High Confidence** – "Significant" indicates that the potential for failure (the combined consideration of the potential for degradation/direct floodwater impact and the implications of that degradation/direct floodwater impact to a structure built to its specific design standard) has been qualitatively evaluated as "high." A description of the reason why a CPFM for any particular structure was placed in this category, including a description of the role each element played in the significance determination, is provided in Sections 5.0 and 6.0 of this

Assessment Report for Priority 1 and Priority 2 Structures, respectively. "High Confidence" indicates that additional information and studies are not likely to increase the confidence in the findings or change the conclusions. The CPFMs included in this category are those for which physical modifications are recommended. Any additional data are required only to facilitate the implementation of those physical modifications.

Document the results using a four-quadrant matrix. This matrix, provided as Table 3-1, shows the rating for the estimated total potential for failure along the vertical axis and the level of confidence along the horizontal axis.

Table 3-1 – Potential for Failure/Confidence Matrix and Associated Recommended Actions

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**	Recommend additional detailed forensic investigations and/or monitoring leading to a decision on physical modification to a structure	Recommend detailed forensic investigations leading to physical modification to a structure
Potential for Failure **Not Significant**	Recommend continued monitoring to confirm no further recommended actions	No further recommended actions related to the 2011 flood

Step 8. **Report** – Following the potential-for-failure assessment, determine whether additional data are needed. Summarize the results of the assessment, and document specific recommended actions.

3.3 Field Observations

The 2011 flood event covered nearly 80 percent of the FCS site. Some of the Priority 1 Structures were protected by engineering measures (such as sandbags, temporary berms, and other flood-proofing measures), but many of the Priority 1 Structures, including a number of buried infrastructure systems, were not. As floodwater receded, visual observations of each structure were conducted to identify any obvious signs of distress or to identify Triggering Mechanisms that could lead to distress. The inspections were completed by three-person teams consisting of senior HDR professionals experienced in structural, civil, and geotechnical engineering. The overall FCS site was also visited by a variety of other professionals for purposes of generally assessing the flood damages and site conditions.

Prior to conducting the site inspections, each discipline lead developed a checklist of specific structural and utility system concerns or issues that might have resulted from prolonged exposure to the floodwater. Copies of each checklist (structural, civil, and geotechnical) are included in Attachment 3 of this Assessment Report. Examples of the concerns and issues include the following:

- Is there evidence of distress from flood forces on the structure caused by foundation uplift, foundation undermining, or other actions?
- Is there evidence of surface erosion or observable scour?
- Is the existing revetment protection undamaged?
- Is there evidence of moisture damage to concrete or metallic surfaces?
- Are there any signs of tilting or cracking of concrete slabs?
- Is there observable ground subsidence?
- Is there observable pavement subsidence?
- Is there observable piping (sand boils, sinkholes)?

3.4 Identified Potential Failure Modes

The assessment teams identified 15 Triggering Mechanisms relative to the 2011 flood and FCS site inundation that could materially and negatively impact structures. Once the Triggering Mechanisms were identified, PFMs that could develop as a result of those mechanisms were identified. A list of identified Triggering Mechanisms and associated PFMs is provided in Table 3-2.

Table 3-2 – Triggering Mechanisms and Potential Failure Modes

Triggering Mechanism No.	Triggering Mechanism	PFM No.	Potential Failure Mode
1	River Bank Erosion/Scour	1a	Undermining shallow foundation/slab
		1b	Loss of lateral support for pile foundation
		1c	Undermined buried utilities pipes/cables
		1d	Additional lateral force on piles
2	Surface Erosion	2a	Undermining shallow foundation/slab
		2b	Loss of lateral support for pile foundation
		2c	Undermined buried utilities
3	Subsurface Erosion/Piping	3a	Undermining and settlement of shallow foundation/slab (due to pumping)
		3b	Loss of lateral support for pile foundation (due to pumping)
		3c	Undermined buried utilities (due to pumping)
		3d	Undermining and settlement of shallow foundation/slab (due to river drawdown)
		3e	Loss of lateral support for pile foundation (due to river drawdown)
		3f	Undermined buried utilities (due to river drawdown)
		3g	Sinkhole development due to piping into karst voids
4	Hydrostatic Lateral Loading (water loading on structures)	4a	Overturning
		4b	Sliding
		4c	Wall failure in flexure
		4d	Wall failure in shear
		4e	Excess deflection

Table 3-2 – Triggering Mechanisms and Potential Failure Modes			
Triggering Mechanism No.	**Triggering Mechanism**	**PFM No.**	**Potential Failure Mode**
5	Hydrodynamic Loading[A]	5a	Overturning
		5b	Sliding
		5c	Wall failure in flexure
		5d	Wall failure in shear
		5e	Damage by debris
		5f	Excess deflection
6	Buoyancy, Uplift Forces on Structures	6a	Fail tension piles
		6b	Cracked slab, loss of structural support
		6c	Displaced structure/broken connections
7	Soil Collapse (first time wetting)	7a	Cracked slab, differential settlement of shallow foundation, loss of structural support
		7b	Displaced structure/broken connections
		7c	General site settlement
		7d	Piles buckling from down drag
8	Soil Solutioning	8a	Not applicable
9	Swelling of Expansive Soils	9a	Cracked slab, differential heave of shallow foundation, loss of structural support
		9b	Displaced structure/broken connections
		9c	Fail tension piles
		9d	Additional lateral force on below-grade walls
10	Machine/Vibration-Induced Liquefaction	10a	Cracked slab, differential settlement of shallow foundation, loss of structural support
		10b	Displaced structure/broken connections
		10c	Additional lateral force on below-grade walls
		10d	Pile/pile group instability
11	Loss of Soil Strength due to Static Liquefaction or Upward Seepage	11a	Cracked slab, differential settlement of shallow foundation, loss of structural support
		11b	Displaced structure/broken connections
		11c	Additional lateral force on below-grade walls
		11d	Pile/pile group instability
12	Rapid Drawdown	12a	River bank slope failure and undermining surrounding structures
		12b	Lateral spreading
13	Submergence	13a	Corrosion of underground utilities
		13b	Corrosion of structural elements
14	Frost Effects	14a	Not applicable
15	Karst Foundation Collapse	15a	Piles punching through karst voids due to additional loading

3.5 Initial Screening of Potential Failure Modes

A summary of Triggering Mechanisms and associated PFMs by structure is presented in Attachment 4. Structures to be assessed were selected and prioritized by OPPD and included buildings, process structures, equipment foundations, tank foundations, and electrical towers (structures). In Attachment 4, the structures are grouped into three categories:

- Class I structures
- Non-class I structures inside the Protected Area
- Non-class I structures outside the Protected Area

PFMs judged by the assessment teams to be credible based on initial screening are labeled "C" in Attachment 4. Failure modes deemed non-credible are labeled NC in Attachment 4, and failure modes that do not apply to a particular structure are labeled NA in Attachment 4.

Attachment 4 presents the results of initial screening. As more information becomes available, each PFM will be reevaluated and rerated as appropriate. The results of the PFM analysis for each structure and system are presented in Section 5.0 of this Assessment Report.

3.6 Potential Failure Modes Deemed Non-Credible for All Structures

The results of the field observations combined with review of FCS design documents indicated that some of the PFMs listed in Table 3-2 were not possible. For example, site investigations revealed no evidence of bank scour along the east boundary of the site. Therefore, failure modes associated with river scour/bank erosion were non-credible. The failure modes described in Table 3-3 were judged to be non-credible for all Priority 1 Structures evaluated. The failure modes described in Table 3-4 were judged to be non-credible for all Priority 2 Structures evaluated.

Table 3-3 – Potential Failure Modes Determined to be Non-Credible for Priority 1 Structures

Identifier	Potential Failure Mode	Rationale for Elimination
Triggering Mechanism 1 – River Bank Erosion/Scour		
PFM 1a	Undermining shallow foundation/slab	• Bathymetric survey of the river channel and banks indicated no observable sloughing, scouring, or other signs of bank erosion. • Visual observations of the river bank indicated no sloughing, scouring, or other signs of bank erosion. • Bank stabilization features installed by USACE are robust, and there is no known major bank failure as a result of 2011 flooding.
PFM 1b	Loss of lateral support for pile foundation	
PFM 1c	Undermined buried utilities pipes/cables	
PFM 1d	Additional lateral force on piles	
Triggering Mechanism 3 – Subsurface Erosion/Piping		
PFM 3g	Sinkhole development (due to piping into karst voids)	Karst voids are filled with water. There is no head differential (gradient) to initiate this type of soil erosion.
Triggering Mechanism 8 – Soil Solutioning		
PFM 8a	Various	Mineralogy of local soils is not susceptible to solutioning.

Table 3-3 – Potential Failure Modes Determined to be Non-Credible for Priority 1 Structures		
Identifier	**Potential Failure Mode**	**Rationale for Elimination**
Triggering Mechanism 9 – Swelling of Expansive Soils		
PFM 9a	Cracked slab, differential heave of shallow foundation, loss of structural support	• Highly expansive soils are not present at the FCS site. Structures are founded either on non-expansive select fill or on non-expansive native granular soils (pile-supported structures). • With respect to soil saturation of expansive soils, the 2011 flood event was not unusual because similar soil wetting occurred during several past floods for the majority of the site.
PFM 9b	Displaced structure/broken connections	
PFM 9c	Fail tension piles	
PFM 9d	Additional lateral force on below-grade walls	
Triggering Mechanism 15 – Karst Foundation Collapse		
PFM 15a	Piles punching through karst voids due to additional loading	Piles were driven or drilled to an elevation below the deepest karst/erosional feature. Explorations for the design/construction extended into bedrock. No voids exist below the pile tips. Additional vertical load due to soil downdrag is minimal compared to the "baseline" vertical load.

Table 3-4 – Potential Failure Modes Determined to be Non-Credible for Priority 2 Structures		
Identifier	**Potential Failure Mode**	**Rationale for Elimination**
Triggering Mechanism 1 – River Bank Erosion/Scour		
PFM 1a	Undermining shallow foundation/slab	River is back to nominal normal levels, and the Triggering Mechanism was not observed.
PFM 1b	Loss of lateral support for pile foundation	
PFM 1c	Undermined buried utilities pipes/cables	
PFM 1d	Additional lateral force on piles	
Triggering Mechanism 3 – Subsurface Erosion/Piping		
PFM 3d	Undermining and settlement of shallow foundation/slab (due to river drawdown)	River is back to nominal normal levels and the PFMs were not observed.
PFM 3e	Loss of lateral support for pile foundation (due to river drawdown)	
PFM 3f	Undermined buried utilities (due to river drawdown)	
PFM 3g	Sinkhole development (due to piping into karst voids)	Karst voids are filled with water. There is no head differential (gradient) to initiate this type of soil erosion.
Triggering Mechanism 8 – Soil Solutioning		
PFM 8a	Various	Mineralogy of local soils is not susceptible to solutioning.
Triggering Mechanism 10 – Machine/Vibration Induced Liquefaction		
PFM 10a	Crack Slab, differential settlement of shallow foundation, loss of structural support	Groundwater is back to nominal normal levels, and the PFMs were not observed.
PFM 10b	Displaced structure/broken connections	
PFM 10c	Additional lateral force on below-grade walls	
PFM 10d	Pile/pile group instability	

Table 3-4 – Potential Failure Modes Determined to be Non-Credible for Priority 2 Structures		
Identifier	**Potential Failure Mode**	**Rationale for Elimination**
Triggering Mechanism 12 – Rapid Drawdown		
PFM 12a	River bank slope failure and undermining surrounding structures	Groundwater is back to nominal normal levels. and the PFMs were not observed.
PFM 12b	Lateral spreading	
Triggering Mechanism 13 – Submergence		
PFM 13a	Corrosion of underground utilities	The structures were not subjected to a corrosive environment that would be considered beyond normal conditions.
PFM 13b	Corrosion of structural elements	
Triggering Mechanism 14 – Frost Effects		
PFM 14a	Various	Prior to ground freezing, the groundwater returned to nominal normal levels.
Triggering Mechanism 15 – Karst Foundation Collapse		
PFM 15a	Piles punching through karst voids due to additional loading	Piles were driven or drilled to an elevation below the deepest karst/erosional feature. Explorations for the design/construction extended into bedrock. No voids exist below the pile tips. Additional vertical load due to soil downdrag is minimal compared to the "baseline" vertical load.

3.7 Assessment Methods

Table 3-5 lists the various methods that might be used to determine the significance of the potential of failure for any of the structures. The methods included visual observations of the structures and civil works, field surveys, and geophysical and geotechnical investigations. Field teams composed of structural, civil, and geotechnical engineering professionals examined the structures as floodwater receded. These investigations were based on detailed checklists, as noted in Section 3.3. The results of the visual observations were supplemented with elevation surveys and geophysical and geotechnical investigations.

Table 3-5 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes					
Potential Failure Mode (PFM)		**Investigation Method**			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
1. River Bank Erosion/Scour	a. Undermining shallow foundation/slab	These PFMs were determined to be non-credible.			
	b. Loss of lateral support for pile foundation				
	c. Undermined buried utilities pipes/cables				
	d. Additional lateral force on piles				
2. Surface Erosion	a. Undermining shallow foundation/slab	[*Note*: these actions were taken for each PFM.] Interview OPPD staff. Review plans and specifications to identify pertinent design and construction details needed to define pre-flood conditions. Review OPPD Condition Reports to determine changes and modifications since construction. Review flood data including observed flow conditions, depths, and velocities.	Observe surface condition for erosion, broken pavement, depressions, gullies, and other signs of distress, and hand probe area adjacent to structures.	Look for settlement of slab, cracks in foundation and walls, tilt, or settlement of foundation.	
	b. Loss of lateral support for pile foundation		Observe soil conditions around structure for settlement.	Observe pile-supported slab for cracking or excessive deflection.	

Table 3-5 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes					
Potential Failure Mode (PFM)		**Investigation Method**			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
2. Surface Erosion (continued)	c. Undermined buried utilities		Observe surface condition for erosion, broken pavement, depressions, gullies, and other signs of distress, and hand probe area adjacent to structures. TV open conduits and pipe in soil if accessible or as possible.		
3. Subsurface Erosion/Piping	a. Undermining and settlement of shallow foundation/slab (due to pumping)		Observe surface condition around buildings for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations or slabs.	Observe settlement of slabs, cracks in foundation, or settlement of foundation.	Test for voids using ground penetrating radar (GPR). Hydro-excavate suspect areas where feasible.
	b. Loss of lateral support for pile foundation (due to pumping)		Observe soil conditions around structure for settlement.		Sample areas adjacent to structures using standard penetration test (SPT) or cone penetration test (CPT) methods as appropriate.
	c. Undermined buried utilities (due to pumping)		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. TV open conduits and pipe in soil if accessible or as possible. Inspect utility manholes (MHs) if possible. Identify MH penetrations that leak; look for sediment in MH bottom and in pumped water.	Observe soil conditions at utilities for settlement or lost soil material.	Test for voids using GPR. Hydro-excavate suspect areas where feasible. Open test pit where feasible.

Table 3-5 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes					
Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	PFM Description	Background Data Research	Field Observations	Structure Assessment	Subsurface Investigations
3. Subsurface Erosion/Piping (continued)	d. Undermining and settlement of shallow foundation/slab (due to river drawdown)		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe soil conditions around structure for settlement of slab, cracks in foundation, or settlement of foundation.	Test for voids using GPR. Hydro-excavate suspect areas where feasible.
	e. Loss of lateral support for pile foundation (due to river drawdown)		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures.	Observe soil conditions around structure for settlement.	Sample areas adjacent to structures using SPT or CPT methods as appropriate.
	f. Undermined buried utilities (due to river drawdown)		Observe surface condition for anomalies, and hand probe alignment of area adjacent to structures. TV open conduits and pipe in soil if accessible or as possible. Inspect utility MHs if possible. Identify MH penetrations that leak; look for sediment in MH bottom and in pumped water.	Observe soil conditions at utilities for settlement or lost soil material.	Test for voids using GPR. Hydro-excavate suspect areas where feasible. Open test pit where feasible.
	g. Sinkhole development (due to piping into karst voids)	This PFM was determined to be non-credible.			
4. Hydrostatic Lateral Loading (water loading on structures)	a. Overturning		Survey/monitor elevation of designated points on foundations.	Observe structures for signs of movement.	
	b. Sliding		Survey/monitor elevation of designated points on foundations.	Observe structures for signs of movement.	

Table 3-5 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes					
Potential Failure Mode (PFM)		**Investigation Method**			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
4. Hydrostatic Lateral Loading (water loading on structures) (continued)	c. Wall failure in flexure		Survey/monitor elevation of designated points on foundations.	Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	
	d. Wall failure in shear		Survey/monitor elevation of designated points on foundations.	Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	
	e. Excess deflection		Survey/monitor elevation of designated points on foundations.	Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	
5. Hydrodynamic Loading	a. Overturning		Survey/monitor elevation of designated points on foundations.	Observe structures for signs of high water exposure or structure movement.	
	b. Sliding		Survey/monitor elevation of designated points on foundations.	Observe structures for signs of high water exposure or structure movement.	
	c. Failure in flexure			Observe exposed structure for signs of high water. Observe exposed structural elements for signs of cracking, water leakage, or excessive (visible) deflection.	

Table 3-5 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
5. Hydrodynamic Loading (continued)	d. Failure in shear			Observe exposed structure for signs of high water. Observe exposed structural elements for signs of cracking, water leakage, or excessive (visible) deflection.	
	e. Damage by debris			Observe exposed structure for signs of high water or impact abrasions/damage from debris.	
	f. Excess deflection			Observe exposed structure for signs of high water. Observe exposed structural elements for signs of cracking, water leakage, or excessive (visible) deflection.	
6. Buoyancy, Uplift Forces on Structures	a. Failed tension piles			Observe pile-supported slabs for cracking, upward deflection.	
	b. Cracked slab, loss of structural support		Observe perimeter grade condition for anomalies, and hand probe alignment or area adjacent to structures.	Observe pile supported slabs for cracking or upward deflection.	Hydro-excavate suspect areas.
	c. Displaced structure/broken connections		Observe perimeter grade condition for anomalies, and hand probe alignment or area adjacent to structures.	Observe structures for cracking, broken members, or other signs of structural distress.	

Table 3-5 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
7. Soil Collapse (first time wetting)	a. Cracked slab, differential settlement of shallow foundation, loss of structural support		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe soil conditions around structure for settlement of slab, cracks in foundation, or settlement of foundation.	Hydro-excavate suspect areas. Obtain undisturbed samples, and test density and water content.
	b. Displaced structure/broken connections		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe structures for cracking, broken members, or other signs of structural distress.	Hydro-excavate suspect areas. Obtain undisturbed samples, and test density and water content.
	c. General site settlement		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.		Hydro-excavate suspect areas. Obtain undisturbed samples, and test density and water content.
	d. Piles buckling from down drag			Observe pile-supported slabs for cracking or downward deflection.	Hydro-excavate suspect areas. Obtain undisturbed samples, and test density and water content.
8. Soil Solutioning	a. Not applicable	This PFM was determined to be non-credible.			

Table 3-5 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes					
Potential Failure Mode (PFM)		**Investigation Method**			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
9. Swelling of Expansive Soils	a. Cracked slab, differential heave of shallow foundation, loss of structural support	These PFMs were determined to be non-credible.			
	b. Displaced structure/broken connections				
	c. Fail tension piles				
	d. Additional lateral force on below-grade walls				
10. Machine/ Vibration-Induced Liquefaction	a. Cracked slab, differential settlement of shallow foundation, loss of structural support		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe foundations for cracking and/or deflection from swelling.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
	b. Displaced structure/broken connections		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe structures for cracking, broken members, or other signs of structural distress.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
	c. Additional lateral force on below-grade walls			Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.

Table 3-5 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes

Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
10. Machine/ Vibration-Induced Liquefaction (continued)	d. Pile/pile group instability			Observe pile-supported slabs for cracking, downward deflection. Test for voids using GPR.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
11. Loss of Soil Strength due to Static Liquefaction or Upward Seepage	a. Cracked slab/differential settlement of shallow foundation/loss of structural support		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe foundations for cracking and/or deflection from swelling. Survey/monitor elevation of designated points on foundations.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
	b. Displaced structure/broken connections		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe structures for cracking, broken members, or other signs of structural distress.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
	c. Additional lateral force on below-grade walls			Observe perimeter walls and below-grade walls for signs of cracking, water leakage, or excessive (visible) deflection.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.

Table 3-5 – Potential Methods and Procedures for Addressing Identified Potential Failure Modes					
Potential Failure Mode (PFM)		Investigation Method			
Triggering Mechanism	**PFM Description**	**Background Data Research**	**Field Observations**	**Structure Assessment**	**Subsurface Investigations**
11. Loss of Soil Strength due to Static Liquefaction or Upward Seepage (continued)	d. Pile/pile group instability			Observe pile-supported slabs for cracking, downward deflection.	Sample areas adjacent to structures using SPT or CPT methods as appropriate. Hydro-excavate suspect areas. Conduct seismic refraction surveys.
12. Rapid Drawdown	e. River bank slope failure and undermining surrounding structures		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures. Survey/monitor elevation of designated points on foundations.	Observe soil conditions around structure for eroded or lost material, settlement of slab, cracks in foundation, or settlement of foundation.	Install and monitor inclinometers. Hydro-excavate suspect areas.
	f. Lateral spreading		Observe surface condition for anomalies, and hand probe alignment or area adjacent to structures.	Observe site soils conditions for signs of soil movements or spreading.	Install and monitor inclinometers.
13. Submergence	a. Corrosion of underground utilities				
	b. Corrosion of structural elements			Observe exposed structural elements for signs of rust, degraded material, or other signs of corrosion.	
14. Frost Effects	a. Not applicable				Test soil properties.
15. Karst Foundation Collapse	a. Piles punching through karst voids due to additional loading	This PFM was determined to be non-credible.			

SECTION 4.0
KEY DISTRESS INDICATORS

4.0 KEY DISTRESS INDICATORS

During the site visual assessments, three problem areas were observed that potentially indicated that the 2011 flood had changed the site's geotechnical and physical character. These observed problem areas, referred to as Key Distress Indicators (KDIs), are the following:

1. Increased groundwater flow into the Turbine Building sump
2. Pavement failure and sinkhole in the paved access area between the Intake Structure and Service Building
3. Column settlement in the Maintenance Shop

The locations of these KDIs are shown in Figure 4-1. Each of the observed KDIs was evaluated using PFM analysis to determine the associated Triggering Mechanism, to identify the CPFMs, to identify other structures that could be affected by the same PFM, and to recommend remedial measures intended to restore the KDIs to their pre-flood condition.

4.1 Increased Groundwater Flow into the Turbine Building Sump

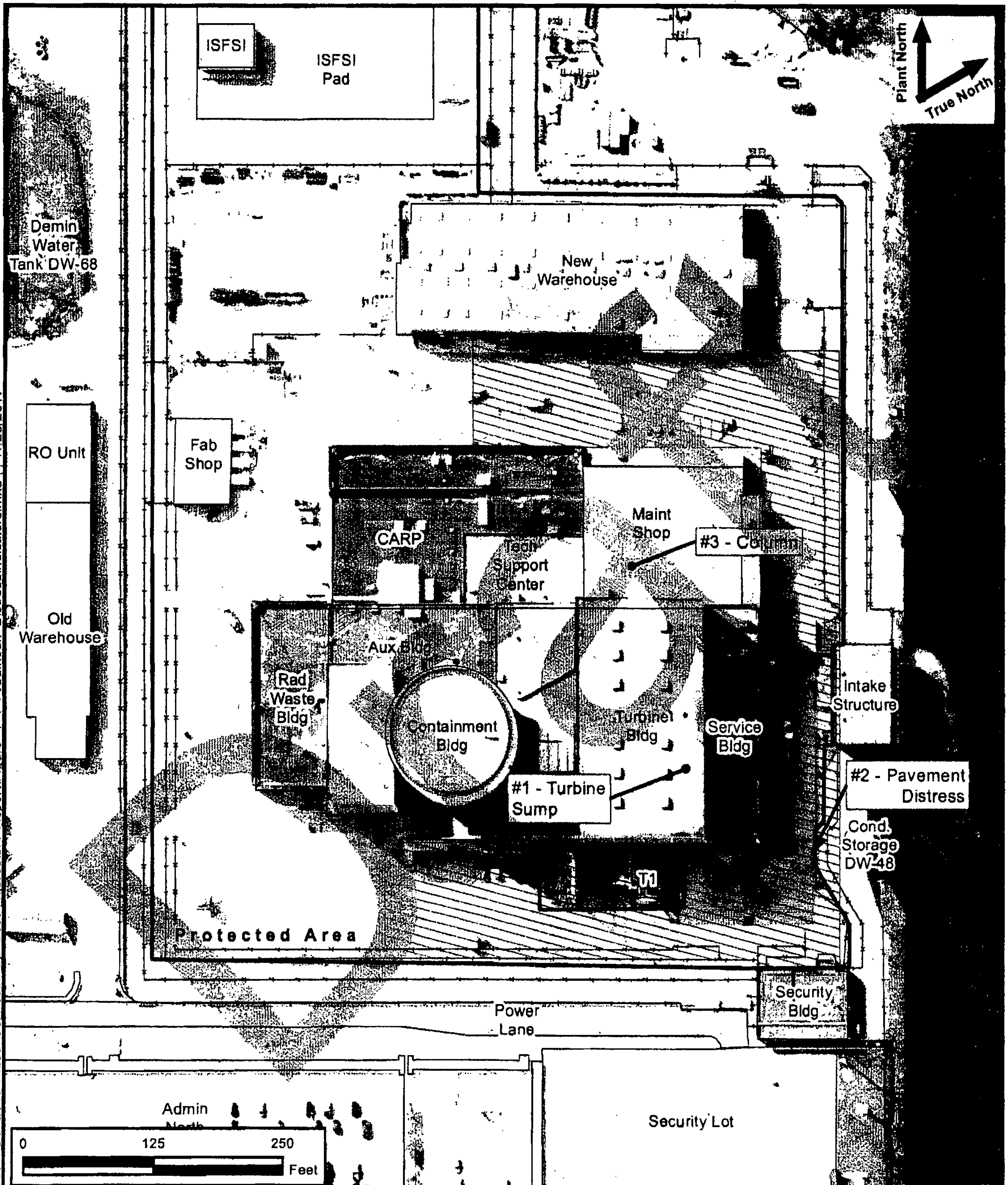
KDI #1 is the increased flow of groundwater (above that anticipated in the original design) into the Turbine Building sump. PFM analysis determined that the Triggering Mechanism associated with increased flow in the Turbine Building sump is Subsurface Erosion/Piping. The three CPFMs associated with this Triggering Mechanism are the following:

- CPFM 3a – Undermining and settlement of shallow foundation/slab (due to pumping)
- CPFM 3b – Loss of lateral support for pile foundation (due to pumping)
- CPFM 3c – Undermined buried utilities (due to pumping)

4.1.1 Physical Observations

The Turbine Building has a documented history of a void below the foundation mat dating back to 1997. This void was discovered via cored holes in the foundation slabs and camera recordings of broken drain piping that lies under the floor slab. Conversations with OPPD personnel indicate that groundwater has been flowing at varying rates through these broken pipes into the sump from that time to the present day. The rate of flow into the sump is directly attributed to the hydraulic head of the groundwater; the observed flow rates have increased as the floodwater elevation increased across the facility. This drain pipe system was designed as a closed system; therefore, the pipes are not surrounded by appropriate filter systems to preclude the transportation of soils from under the slab. It is logical to assume that because the groundwater moves below the foundation and into the broken piping, some movement of the soil has occurred.

The increased flow is originating from breaks in the pipes that are designed to carry water from the floor drains in the basement of the Turbine Building. These drains are also used to drain equipment in the Turbine Building. The structures potentially affected by the CPFMs associated with the Triggering Mechanism (Subsurface Erosion/Piping) for Key Distress Indicator #1 are presented in Section 4.1.1. A complete description of each structure is presented in its respective subsection of Section 5.0.



Fence

Area of Concern

Omaha Public Power District

Distress Indicators
Fort Calhoun

Plant and Facility Geotechnical and Structural Assessment

HDR

DATE Sep 2011

FIGURE 4-1

Z:\Projects\OPPD\164565_FCS_2011_Flood_Services\Map_Docs\Figures\SA_Report\Figure_4_1_FtCalhoun_DistressIndicators.mxd , 11/28/2011

The following information was taken from a summary report prepared by OPPD dated March 24, 2009, regarding broken floor drain pipes:

- There are two drain lines that run parallel to each other: the 6-in. floor drain and the 10-in waterbox drain. The drain lines are not cross-connected, so both lines must have a piping break if the 10-in. line is causing the floor drains to back up.
- A vendor was brought in to visually inspect the drain lines. The vendor found a break in the 10-in. drain at the branch tee from the VD-193 drain valve but could not inspect the 6-in. floor drain because the line does not have a cleanout connection in this area and accessibility through floor drains is restricted by a drain trap at each location.
- A review of system files shows that a break in the waterbox drain line has been known about for quite some time. In 1997, a repair was attempted by core drilling holes in the vicinity of the leak and pressure grouting to seal the leak. Per the "Water Systems Report Card for Report Period April 1 Through June 30, 1997" (memo PED/EOS SYE 97-123):

  > Repair of the Turbine Building Basement Drain line header was attempted during this period. The repair procedure consisted of core drilling holes in the vicinity of the leak and pressure grouting to seal the leak. Approximately 10 holes were drilled and it was estimated that a void of approximately 10x8x1 feet existed under the concrete slab. The void was filled with cement grout but the leak could not be stopped. Boroscope inspection of the pipe exterior performed through the core drills showed considerable pipe damage, in more than one location. The extent of the damage and concern over collapsing the line were determining factors in terminating the pressure grouting operation. FC ECN 97-213 was originated to request that a new drain header be installed.

- The grout was injected in the area by the VD-193 (FW-1A south return box tail valve). At some time later, the Turbine Building sump was cleaned out, and a slab of hardened grout was found in the sump, confirming that the grout had flowed through the drain system into the sump. A recent inspection of the floor drains revealed a considerable amount of grout in the floor drain south of the FW-3 Condensate Cooler. The drain looks to be almost fully restricted. This grout most likely came from the 1997 effort, indicating that both lines were broken at that time too.

4.1.2 Triggering Mechanism

As discussed previously, the Triggering Mechanism for increased flow into the Turbine Building sump is Subsurface Erosion/Piping. Multiple potentially connected seepage paths could exist in the soil backfill at the site, including soil backfill in utility trenches, granular trench bedding, and building floor drains with open/broken joints. The paths could be exposed at some locations to the river floodwater and high groundwater. This network of seepage paths could be connected to the sump in the Turbine Building. The breaks in the piping have been documented for an extended period of time (dating back to at least 1997), maintaining a head differential on the potential seepage path networks. The gradient during the 2011 flood was increased, which could have led to higher flows through the seepage path networks. The unfiltered seepage condition will continue until the breaks in the piping system are repaired, which means the potential for further erosion remains. Erosion could extend out, creating voids under other structures.

Review of video from the sump and visual observations indicate groundwater flowing from all five drains. Drain lines are located below the mat foundation slab. OPPD personnel indicated that the drain lines were cleaned in 2011.

Three soil borings (Boring B22, B-24, and B70) were completed within the Turbine Building footprint as part of the Dames & Moore 1968 report, "Foundation Studies." Excavation for the Turbine Building foundation extended to el. 985 ft, so material logged in these three borings from el. 985 to 975 ft is of most importance to the Key Distress Indicator. Boring 24 (B-24) logged fine sand with clayey silt and silty clay lenses and SPT N-values of 7 and 11; B-22 logged fine sand with some medium sand and SPT N-values of 11 and 7; and B-70 logged fine sand with some medium sand and SPT N-values of 26 and 15. The fine sand is susceptible to piping if water velocity is sufficient. The zones of silty clay and clayey silt encountered in B-24 are the materials most susceptible to piping. Excavation beneath the Turbine Building footprint is shown in the drawing "Excavation and Grading Cross Sections" to extend to an approximate elevation of 984 ft for the overall foundation. Elevations of 979.2 ft were reached for the Sump Pit. Soil density tests reported by Nebraska Testing in 1968 during foundation preparation show density test elevation for the Turbine Generator Mat as low as el. 977 to 980.2 ft and ranging from 97 to 100 percent of modified (specifications require 95% modified) using the American Association of State Highway and Transportation Officials (AASHTO) test method T-147-54 for the Turbine Generator Mat. Material was described as brown sand. The test elevations below the excavation level of 984 ft likely indicate a zone between the piles overexcavated due to the presence of loose material.

The portion of the drain pipe located below the foundation slab consists of asbestos-cement sewer pipe. The material placed around the pipe is assumed to be sand from the excavation. No density tests of the backfill around the drain pipes are available.

This review of the data associated with the Turbine Building foundation and drain piping confirm that the Triggering Mechanism Subsurface Erosion/Piping due to pumping (void development) is a plausible scenario.

4.1.3 Structures and CPFMs Associated with Triggering Mechanism

The Triggering Mechanism identified for increased flow into the Turbine Building sump also applies to the following structures and CPFMs:

- Containment
  - CPFM 3b – Subsurface Erosion/Piping. Loss of lateral support for pile foundation (due to pumping).
- Rad Waste Building
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Technical Support Center
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Turbine Building
  - CPFM 3b – Subsurface Erosion/Piping. Loss of lateral support for pile foundation (due to pumping).
- BBREs
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Turbine Building South Switchyard
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).

- Main Underground Cable Bank (Inside and Outside the PA)
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Circulating Water System
  - CPFM 3b – Subsurface Erosion/Piping. Loss of lateral support for pile foundation (due to pumping).
- Demineralized Water System
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Raw Water Piping
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Fire Protection System Piping
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Service Building
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Maintenance Shop
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).

The influence of this Triggering Mechanism might be somewhat limited by the fact that vibroflotation of subsoils was conducted during construction of the Auxiliary Building and Containment. However, subsurface erosion cannot be completely ruled out because that Triggering Mechanism could extend to the Containment and the Auxiliary Building.

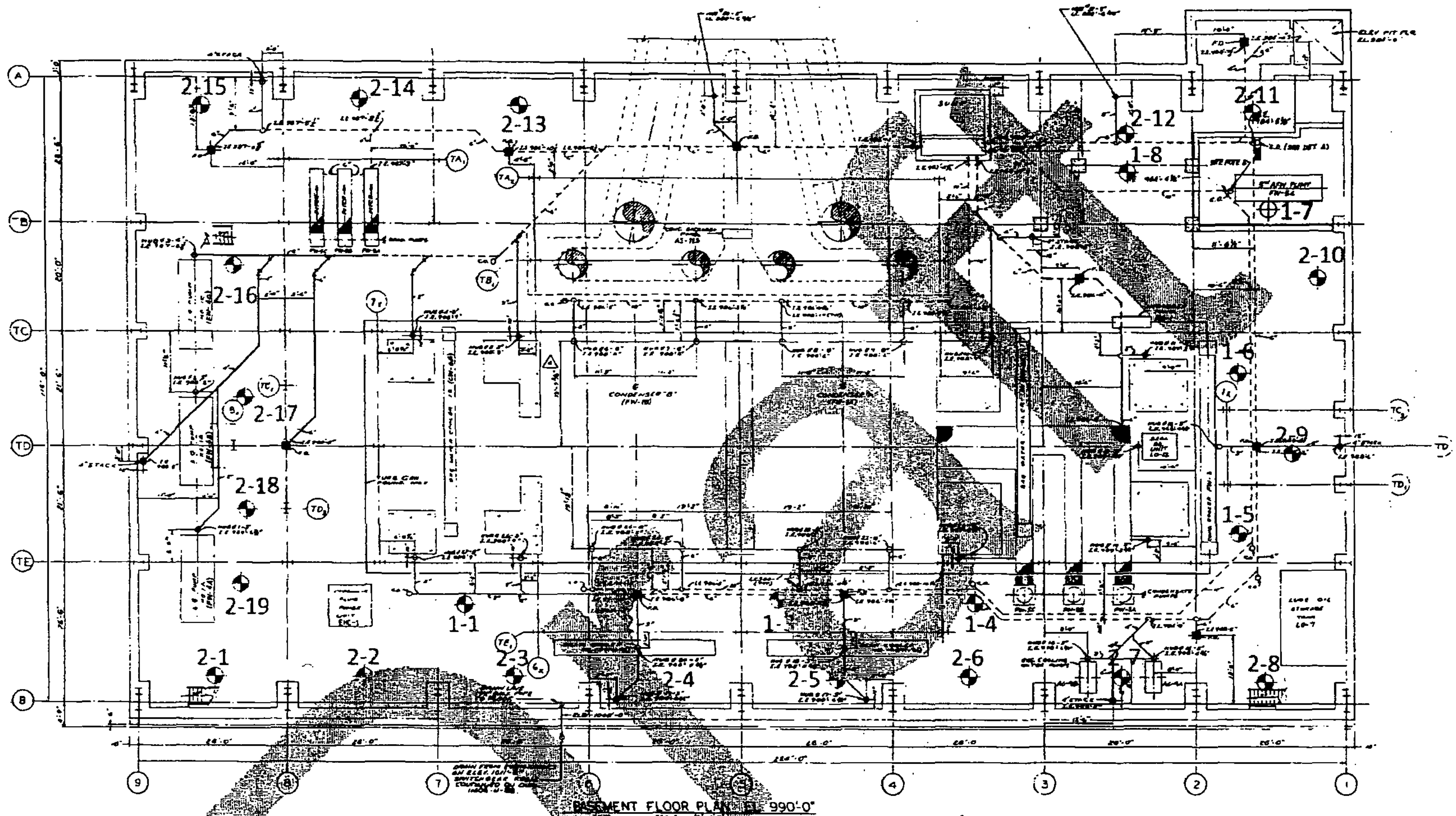
4.1.4 Assessment Methods and Procedures

The following actions are recommended for Key Distress Indicator #1 (increased flow into the Turbine Building sump) and Triggering Mechanism (Subsurface Erosion/Piping due to pumping).

4.1.1.1 Concrete Drilling and Sub-grade Testing Program

To determine the presence and vertical extents of potential voids within the slab subsurface, a procedure to conduct the drilling of selective holes in the Turbine Building basement floor is proposed.

Preliminary locations for the proposed drill holes are shown in Figure 4-2. Eight holes were located in areas of anomalies as identified during Ground Penetrating Radar (GPR) testing and analysis. Nineteen holes were distributed around the Turbine Building basement perimeter to further explore for voids and determine their connection with surrounding structures. The locations shown in Figure 4-2 are preliminary and approximate. A detailed drilling plan will be developed based on a site examination with the appropriate OPPD personnel to determine locations that minimize impacts on the structure, underground piping, and equipment.



Notes:
1. Drilling locations 1-x are located near anomalies as identified in GPR analysis.
2. Drilling locations 2-x are spread along the basement perimeter for void exploration.
3. Drilling location 1-7 has been removed due to equipment conflicts.

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FIGURE
4-2

Anticipated requirements for the drilling and testing program include the following:

1. Establish concrete thickness during core tests for future reference.
2. Observe water flowing out of drill hole.
3. Determine vertical dimensions of any voids found immediately below the slab.
4. Conduct miniature cone or other penetration tests for up to 10 ft below the bottom of the slab to determine soil density.
5. Obtain and test soil samples.
6. Insert a borescope through the drill hole if a void is detected.

Recommended Drilling and Inspection Procedure – The drilling locations should be marked using a rebar locator to adjust drilling location as needed. Care should be taken to avoid conflicts with the drainage system below the floor slab. Additional data on the drainage system from recent inspections should be used.

A carbide-tipped hammer drill should be used to drill a 1-in.-diameter inspection hole at locations specified in the detailed drilling plan. The floor at the drilling locations is expected to be 2 ft 7 in. thick. A means and method of temporarily plugging the hole while under pressure from groundwater shall be proposed by the drilling sub-contractor and approved by OPPD and HDR staff prior to initiation of the drilling operations. Casings grouted into the foundation mat could be required in order to achieve top-of-casing elevations sufficient to prevent flow of water into the Turbine Building due to artesian pressure.

A miniature cone designed to record tip resistance should be pushed into the foundation soil material 10 ft beneath the foundation mat in order to record tip resistance. Depths of voids and soft zones encountered will be determined and documented. In addition to the use of a miniature cone, a waterproof borescope with lighting should be available to investigate further and determine extents of any voids encountered.

Upon completion of inspection, the floor holes will be refilled with non-shrink grout having a minimum 28-day compressive strength of 5000 psi. Repair of holes shall meet OPPD criteria and be approved by OPPD staff.

Upon completion of the proposed drilling and inspection work, OPPD and HDR staff will discuss the necessity for and location of additional drilling locations, possibly to include adjacent buildings, to further define the extent of any voids encountered. Approval by OPPD staff will be required prior to beginning any additional drilling operation.

4.1.4.1 Recommendations Related to Other Key Distress Indicators

Additional forensic investigations associated with Key Distress Indicators #2 (Pavement failure and sinkhole in the paved area between the Intake Structure and Service Building) and #3 (Settled Column in Maintenance Shop) could increase the confidence in the determination of the significance of the potential for degradation associated with this Triggering Mechanism (Subsurface Erosion/Piping due to pumping). See Sections 4.2 and 4.3 for details on the recommendations for additional forensic investigations associated with KDIs #2 and #3, respectively.

4.1.4.2 Recommended Physical Modifications

Repair Drainage Pipes – Repair of the drainage pipes to stop the groundwater flow into the sump pit from the breaks in the drain piping is critical. This could be accomplished by repairing the breaks in the existing pipes or by plugging the existing pipes and constructing a new drain system.

Treatment of Voids Beneath the Turbine Building Foundation – If voids are encountered during the concrete drilling and sub-grade testing program, a grouting program should be designed and completed to fill the voids and restore the integrity of the subgrade. Breaks in the drainage pipes must be repaired prior to initiation of the grouting program.

Depending on the size and extent of the voids, a grout mix and grouting procedure must be developed in order to maximize the stability and effectiveness of the grouting program. The grout mix must be designed to allow maximum migration of the grout past the void limits and into the sandy backfill material. This would reduce seepage potential to any future pipe breaks and will most effectively repair the foundation. The grouting process must be designed to consider the maximum grouting pressure allowed by pipe strength. Grout volumes and grouting pressure must be monitored in real time in order to protect the drainage pipes and any other structures that could be affected by the grouting process.

Monitoring and documentation of the grouting process would provide the data necessary to evaluate the success of the void treatment effort.

4.1.4.3 Continued Monitoring Program

Continued monitoring is recommended to include visual inspections of the structure and surrounding grades and a continuation of the elevation surveys of the previously identified targets on the structures and surrounding site. The purpose is to monitor for signs of structure distress and movement or changes in soil conditions around the structures listed in Section 4.1.1. The results of this monitoring will be used to increase the confidence in the assessment results. Elevation surveys and visual inspections should be performed weekly for 4 weeks and biweekly until December 31, 2011. If any new distress indicators are observed between inspection intervals or after December 31, 2011, appropriate personnel should be notified immediately to determine whether an immediate inspection or assessment should be conducted.

4.1.5 KDI #1 Forensic Investigation

A forensic investigation program consisting of concrete floor slab drilling and subgrade testing was completed in the Turbine Building basement to evaluate subsurface conditions for KDI #1. KDI #1 consists of the increased volume of water pumped from the Turbine Building Sump that has entered the drain pipes through existing breaks in those pipes. The Triggering Mechanism associated with this distress indicator is #3 - Subsurface Erosion Piping and the related CPFMs are 3a, 3b, and 3c, which are all "due to pumping." The flow into the broken drain pipes has caused a cone-of-depression in the groundwater similar to what would have occurred due to the pumping of groundwater from a well (see Figure 4-4). The resulting flow through the subsurface soils into the broken pipes and then into the sump resulted in the piping of soil material out from under the floor slab in the basement of the Turbine Building, and possibly from the subsurface below adjacent Structures. The voids under the Turbine Building Basement floor slab were first observed in 1997 and remedial actions were taken by

OPPD to grout the voids, but were unsuccessful in that the entrance of significant amounts of groundwater into the drainage pipes was not arrested. The purpose of this investigation was to confirm piping and erosion of foundation materials and to estimate the location and possible extent of the void or subgrade disturbance beneath the floor slab and to ascertain their significance related to the CPFMs identified for the Turbine Building itself, and other structures that may be affected by the voids.

4.1.5.1 Scope of Work

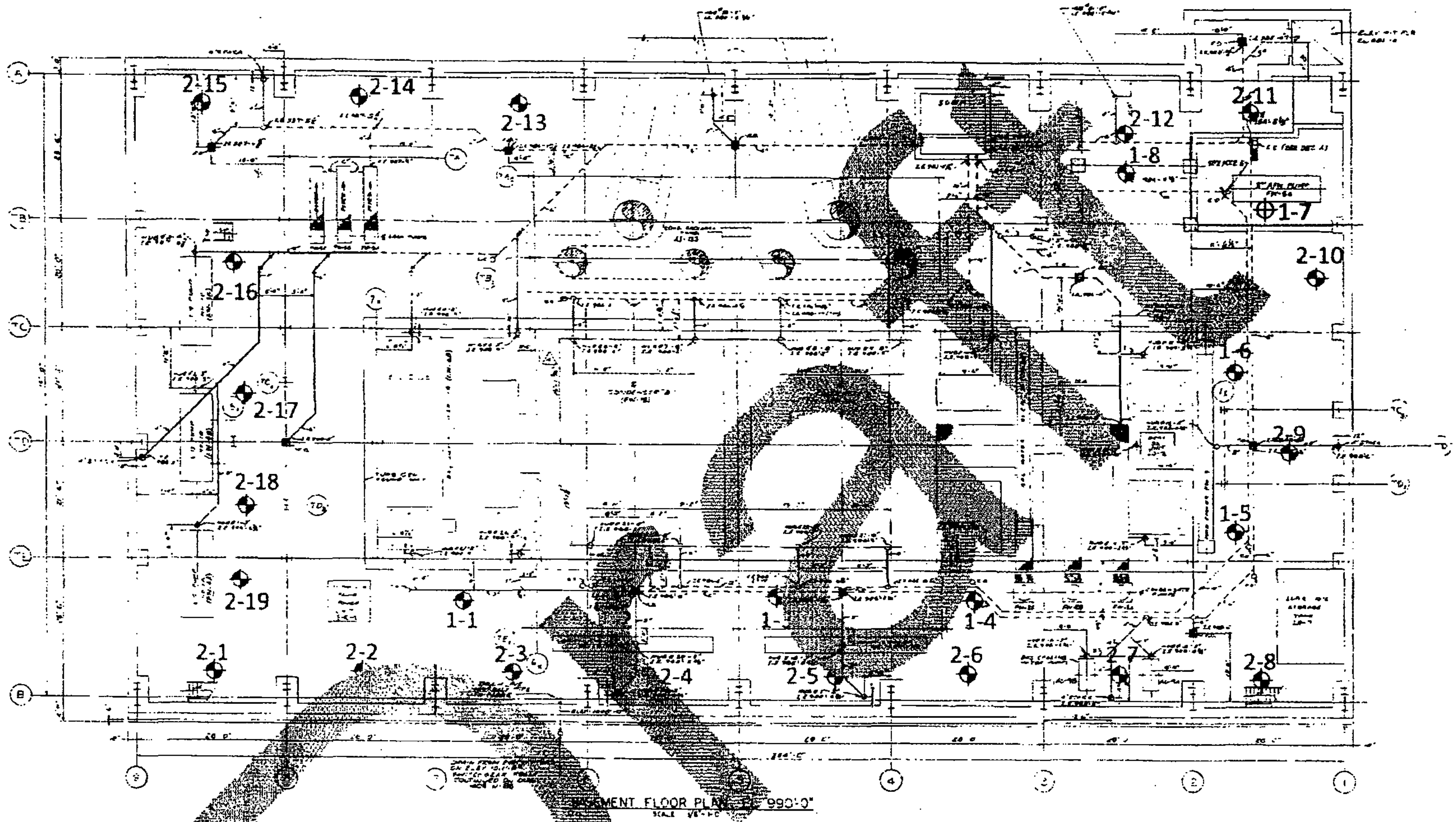
This phase of the forensic investigation of the Turbine Building Basement subgrade began on November 11, 2011. A total of 27 floor slab locations were designated for drilling and subsequent underlying subgrade evaluation (see Figure 4-3 for drill-hole locations). One of the locations, 1-7, was not drilled due to OPPD Plant Safety concerns. Initial investigation of the Turbine Building Basement subgrade and potential piping began in August 2011 with estimates of flow into the sump, GPR surveys performed by Geotechnology, Inc. of the foundation slab and subgrade, and drainage pipe video investigation by Elite Pipeline, Inc. The GPR surveys noted some anomalous zones of seemingly lower density material and the drainage pipe video noted two breaks in the 10" diameter drainage pipe. Drill-hole locations 1-1 through 1-8 as presented in Figure 4-3 were located to investigate these lower density zones and pipe break locations. Drill-hole locations 2-1 through 2-19 were located to provide as much data as possible near the edges of the foundation slab in order to assess the extent of any possible soft zones or voids away from the drainage pipes and sump locations. The Geotechnology Inc. report and Elite Pipeline report are presented in Attachment 6.

Drilling was accomplished by Omaha Concrete Sawing and Lueder Construction Company under contract to OPPD using a hammer drill to advance 1-in. diameter holes in the floor slab through which subgrade evaluations were performed. All drill-holes were covered immediately following drilling and before and after subgrade evaluations using temporary plastic caps that were flush with the surrounding floor surface.

The subgrade evaluation included observation of conditions immediately below the floor slab and then field testing of the subgrade materials at each drilled location. Observations were made by HDR and Thiele Geotechnical, Inc., a geotechnical engineering and testing firm based in Omaha, NE. Subgrade field testing was performed by Thiele as a subcontractor to HDR with HDR representatives present.

Investigation of the subgrade below the floor slab included the following:

- Direct visual observation through the open holes with the aid of a flashlight
- Direct visual observations using a lighted, water-proof bore scope lowered through the open drill-holes
- Estimation of depth to water in each borehole using a T-rod probe
- Measurement of the floor slab thickness
- Depth to subgrade using a tape measure (to determine thickness of existing voids)



Notes:
1. Drilling locations 1-x are located near anomalies as identified in GPR analysis.
2. Drilling locations 2-x are spread along the basement perimeter for void exploration.
3. Drilling location 1-7 has been removed due to equipment conflicts.

Omaha Public Power District

Turbine Building Drilling Locations
Fort Calhoun Station

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FIGURE
4-3

Subgrade testing consisted of dynamic cone penetrometer tests (DCP) at each drilled location. Thiele used a Humboldt Model H-4219 Heavy Duty Dual Mass Dynamic Cone Penetrometer to perform the DCP test in accordance with ASTM D6951/D6951M, Standard Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications. The preferred methods of estimating density of non-cohesive soils is the use of the Standard Penetration Test (SPT) and the Cone Penetrometer Test (CPT). Employment of the SPT and CPT was not possible in this case due to access and space constraints in the Turbine Building basement. The data obtained from the DCP can be used to identify zones of relatively low density or softness compared to the surrounding subgrade as stated in Note 1 of the Standard. Identification of such zones is relevant to developing a comprehensive model of subgrade deterioration due to pumping and material piping.

4.1.5.2 Results

Investigation results are summarized below. The Thiele Geotechnical, Inc. report of DCP testing and DCP logs are presented in Attachment 6.

4.1.5.2.1 Drill-hole Results

Visual observations and measurements were made as described above. Data obtained at each drill-hole are summarized below in Table 4-1. These data are the basis for the top of subgrade surface presented graphically in Figures 4-6 and 4-7.

Table 4-1 - Turbine Building Basement Subgrade Investigation Observations

Drill-hole Number	Floor Slab Thickness (inches)	Initial Depth to Subgrade (inches)	Depth to Subgrade (inches) at DCP Testing (inches)	Initial Potential Void Space Depth (inches)	DCP Depth Tested below floor (ft.)	DCP Elevation Tested (ft.)	Comments
1-1	30.5	31	31	0.5	10	980	water at 22.5" below floor
1-2	32.25	35	35.25	2.75	13	977	air under pressure upon penetrating slab, diminished within 60 seconds
1-3	35.5	38.5	36.25	3	13	977	some air under pressure upon penetrating slab
1-4	38	44	41	6	13	977	
1-5	33	37	33.75	4	10	980	
1-6	31	36	37	5	17	973	pressurized air, then bubbling water 60 seconds after drilling
1-7	NA		NA	NA	NA		Not drilled
1-8	31.25	32	32	0.75	10	980	GW at depth of 12.5", no voids
2-1	30.25	30.75	30.75	0.5	10	980	water 22" below floor
2-2	27.75	33	28.75	5.25	10	980	
2-3	29.5	36	30.75	6.5	10	980	air under pressure

Table 4-1 - Turbine Building Basement Subgrade Investigation Observations

Drill-hole Number	Floor Slab Thickness (inches)	Initial Depth to Subgrade (inches)	Depth to Subgrade (inches) at DCP Testing (inches)	Initial Potential Void Space Depth (inches)	DCP Depth Tested below floor (ft.)	DCP Elevation Tested (ft.)	Comments
							upon penetrating slab,odor of fuel, water bubbled to surface for up to 60 sec.
2-4	30	37	30.5	7	10	980	water 25" below floor
2-5	32.25	34.75	32.25	2.5	13	977	water 25" below floor
2-6	35	38	37	3	13	977	significant soft zone/void
2-7	31.75	33.5	33.5	1.75	10	980	water 27.5" below floor, some air released when slab penetrated
2-8	34.5	36		1.5	13	977	
2-9	31.75	33	33.25	1.25	10	980	water 6" below floor
2-10	31.75	35	32.75	3.25	13	977	
2-11	32.75	35	33.75	1	13	977	
2-12	NK	33	30.75	?	10	980	
2-13	30.75	33.5	31	2.75	17	973	GW 2" below floor
2-14	31.25	34	32.5	2.75	10	980	pressurized air, then water to 10 inches below floor minutes later
2-15	32.25		32.75	1	10	980	GW extruded onto floor intermitantly for 60 sec, then GW 17.25" below floor
2-16	34.25	36	34.75	1.75	13	977	
2-17	30.25	32.5	30.75	2.25	10	980	GW 17.5" below floor
2-18	30.5	32.5	31.25	2	10	980	
2-19	27	31	28	4	10	980	GW 17.5" below floor

As indicated by the measurement data above, floor slab thickness at the locations drilled ranged from 27 to 38 inches. Construction drawings show the floor slab thickness as 31 inches. These differences from the drawings may be attributed to variations during construction. Upon penetration of the slab, the hammer drill often punched through the bottom of the slab and penetrated the subgrade before the drill operator could stop the drill. For this reason, voids between the bottom of the slab and the subgrade were assumed to have developed if the difference between the bottom of the slab and the surface of the subgrade was greater than or equal to 2inches. Void thickness greater than or equal to 2 inches was detected below the floor slab at 16 drilled locations. Overall, void thickness ranged from 2 to 11 inches. Voids were measured immediately after the hammer drill was extracted from the drill-hole.

The void space immediately below the slab was also measured by Thiele immediately prior to DCP testing. In many cases, the void space measured at that time was significantly less than that measured by HDR immediately following foundation slab drilling. The explanation for this discrepancy is that the Thiele measurements were taken hours or even days after the initial drilling occurred, allowing the fine grained silty sand to flow into the void space. Pressurized air was noted flowing from the drill holes at a number of the locations. Once pressure under the foundation slab was released, the groundwater with fine grained silty sand was able to flow into the void space as groundwater was no longer held back by air trapped under the foundation slab. In no case did we encounter evidence of silty sand flowing up into the bore holes.

Figure 4-6 shows the estimated top of subgrade beneath the floor slab based on Probe Measurements taken on 12/09/11 by HDR personnel. Figure 4-7 also presents the estimated top of subgrade beneath the floor slab based on the acquired depth to subgrade data at the time of DCP testing.

4.1.5.2.2 Groundwater Levels

Groundwater was measured in 11 of the borings immediately after drilling and ranged from 2.0 to 27.5 inches below the floor elevation of 990 immediately after foundation slab drilling. Water elevations were not measured in the remaining 16 drill-hole locations due to either dry conditions, difficulty assessing an accurate water elevation due to water level fluctuation, or drill cuttings mixed with water on the drill-hole walls smearing along the T-rod probe and preventing an accurate measurement. In the case of drill-holes 1-6 and 2-15 water flowed from the drill-hole intermittently for approximately one minute then flow ceased. These are the only two cases in which groundwater reached the foundation floor surface. At no time did any water flow onto the foundation floor for any extended period of time. Figure 4-5, Turbine Building Groundwater Gradient Map, shows groundwater contours based on water level data obtained on December 9, 2011. Figure 4-5 shows a fairly consistent groundwater level with the exception of the higher gradient on the south wall. Figure 4-4, Comprehensive Groundwater Gradient Map shows the groundwater gradient in the vicinity of the Turbine Building and adjacent areas. Figures 4-4 and 4-5 show groundwater gradient dropping in elevation toward the southwest corner of the Turbine Building near drill locations 1-3, 1-4, and 2-6. This is consistent with the presence of significant void space in this area and with previous reports (Elite Pipeline Report) that identify drainage pipe breaks and high flow rates in this vicinity. The combination of slightly depressed groundwater elevations and evidence of voids is subjective evidence that subgrade piping due to pumping is occurring in this area. Groundwater contours were generated using MicroStation GeoPak DTM Tools to triangulate between the groundwater elevation points and develop a groundwater elevation surface. From this surface the contours were generated from elevations along the triangulation lines in MicroStation. This function is within standard practices for ground surface and groundwater surface contouring.

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Comprehensive Groundwater
Gradient Map - Contours based on data from 12/09/11
Turbine Building Drilling & Site Monitoring Wells
Fort Calhoun Station

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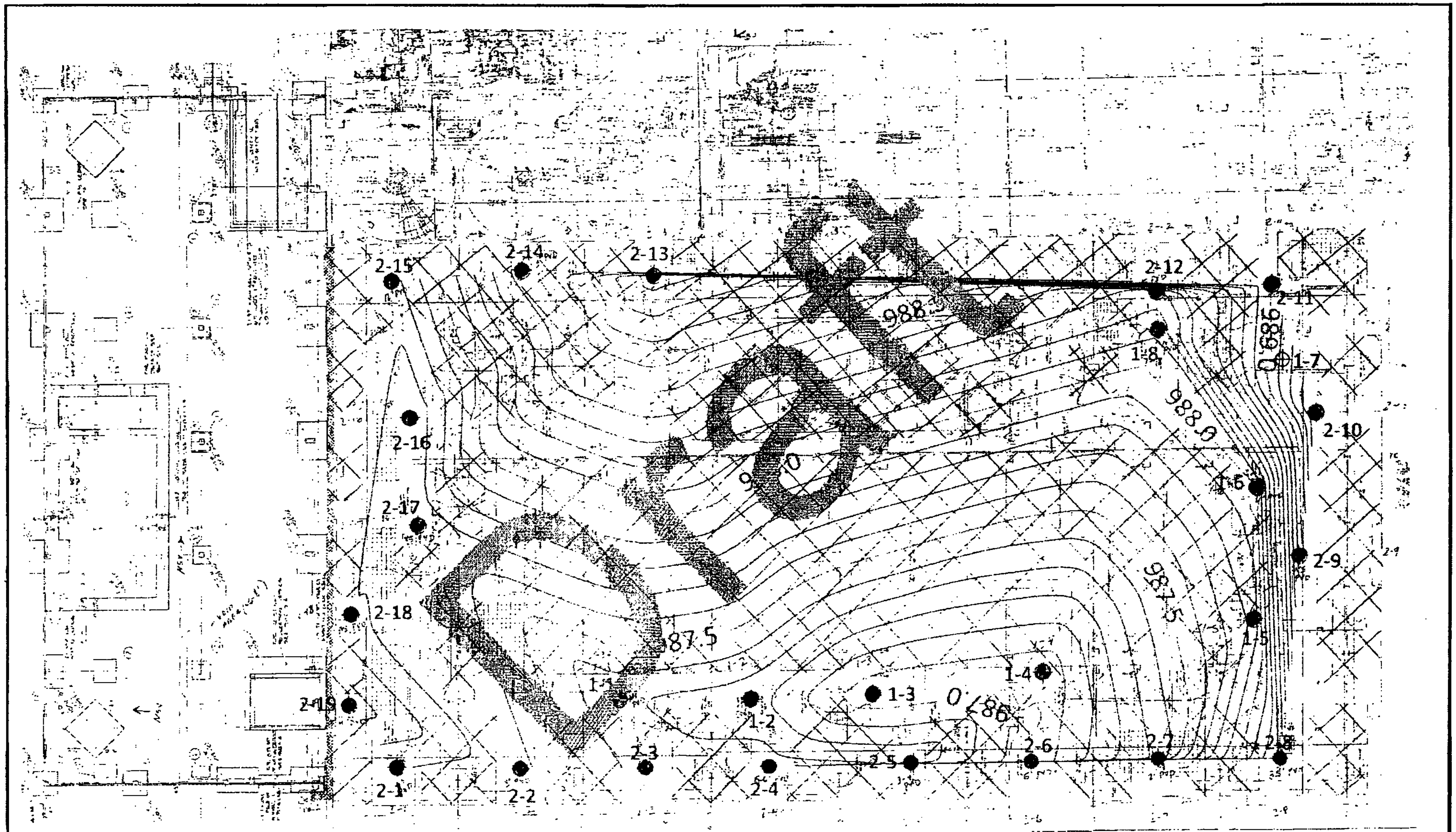
Dec 2011

FIGURE
4-4



987.5 Major Groundwater Contour

Minor Groundwater Contour

X-X Drilling Locations

Note: Groundwater elevations were recorded on 12-9-2011.
Turbine Basement slab elevation is 990.0.

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Turbine Building Ground Water Gradient Map
Fort Calhoun Station

Drilling and Testing Program

HDR

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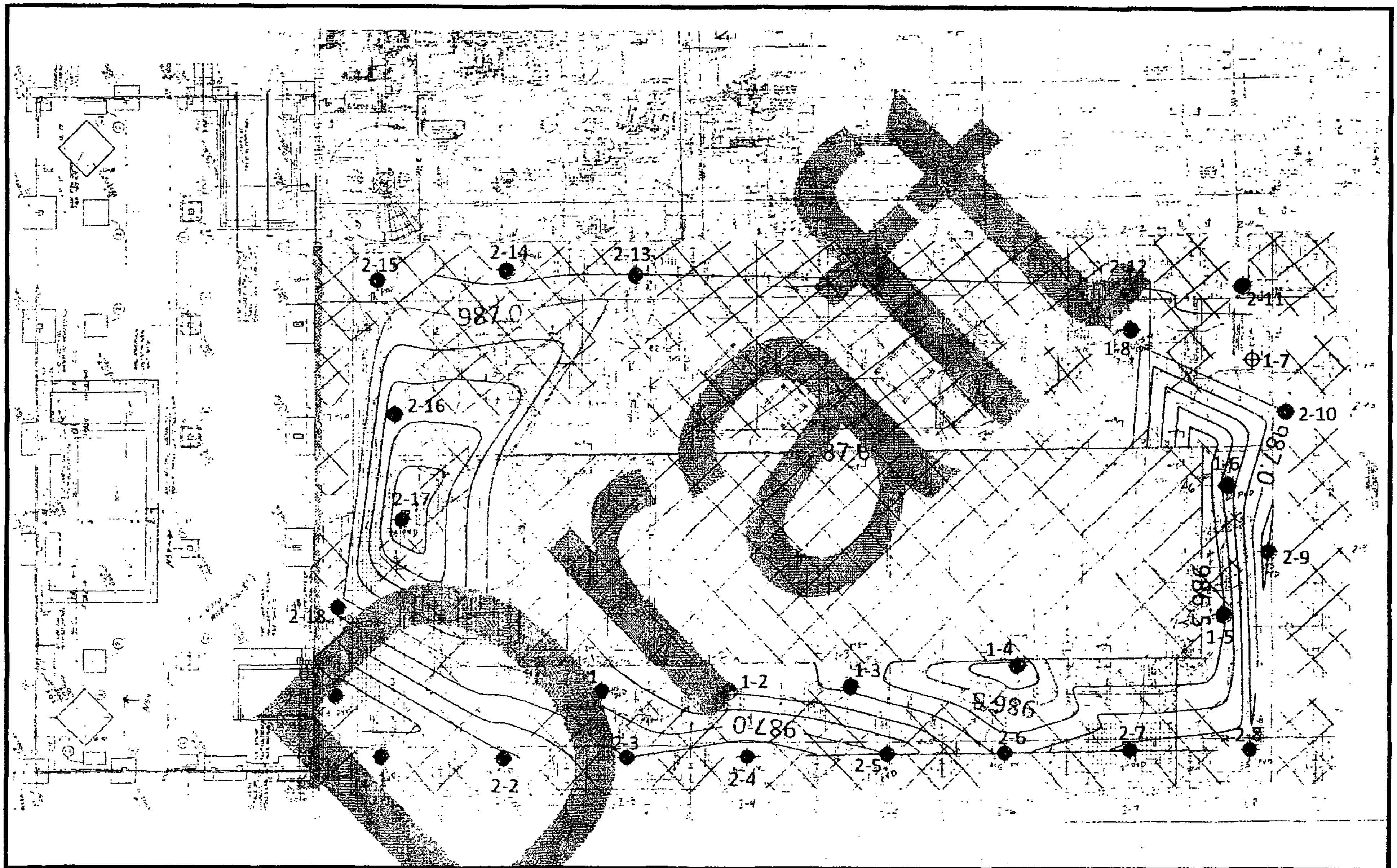
FIGURE: 4-5



--- 987.0 --- Major Subgrade Contour (.5')
——— Minor Subgrade Contour (.1')

X-X Drilling Locations

Note: Subgrade elevations represent probed depth to subgrade as recorded on 12-9-2011.

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Top of Subgrade Topographic Map
Based on Probe Measurements - 12/09/11
Fort Calhoun Station

Plant and Facility Geotechnical and Structural Assessment

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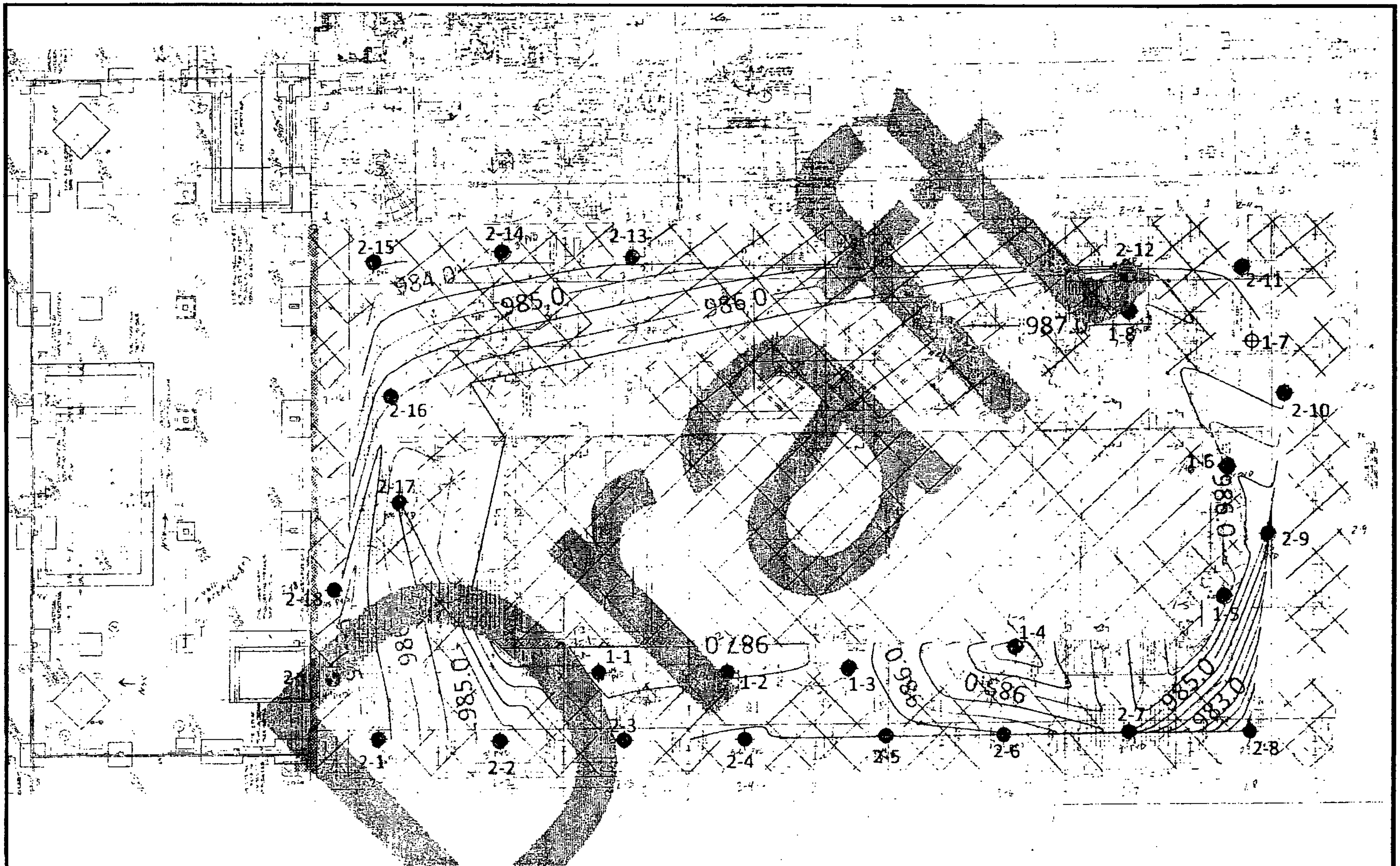
FIGURE	4-6



X-X Drilling Locations

— 987.0 — Major Subgrade Contour (1')

——— Minor Subgrade Contour (.5')

Note: Subgrade elevations represent DCP Test subgrade elevations.
Only shallow voids shown.

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Top of Subgrade Topographic Map Based on Dynamic Cone Penetrometer Tests - 11/16/11 through 12/02/11 Fort Calhoun Station

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FIGURE

4-7

4.1.5.2.3 DCP Test Results

DCP testing began on November 16, 2011 and was completed on December 2, 2011. Thiele performed the DCP with a 10.1 pound hammer recording blow counts for every 2 inches of cone penetration, or as close to each 2 inch penetration interval as possible. The data and related calculated bearing capacity in pounds per square foot (psf) and pounds per square inch (psi) is presented in Attachment 6 - DCP Field Test Data.

The DCP test is useful in indentifying zones that are soft or loose (with low blow counts) relative to the surrounding fill as noted in ASTM D6951/D6951M, Note 1. Any zone that drove with the weight of the hammer without a drop of the hammer (zero blow count zone) and/or with only one blow is considered a very loose zone that has been filled in with flowing sand. In addition, any zone that drove more than 2 inches with only 2 blows is considered soft and likely altered by some process of material loss but is still considered soil. Previous density test of fill in the vicinity of the Turbine Building during construction range from 87 to 105 percent compaction and N values of in situ soil under the Turbine Building from preconstruction borings yielded SPT N values of no lower than 2 blows per foot and commonly 4 to 11 blows per foot in the upper soil zone. There were no cases of weight of rod material in original borings. As noted previously there is no direct correlation of DCP to SPT tests, but it is our engineering opinion that material that allows a tip to drive through soil under weight of rods or weight of rods and hammer provides evidence of extremely soft material that does not reflect conditions at the time of construction. These voids and soft zones do not include the void space between the foundation slab and the top of subgrade presented in Table 4-1 since the DCP tests began at the top of subgrade.

A number of zero blow count zones and soft zones were identified using the DCP. Twenty-one of the 26 DCP tests showed some void within the zone of tested soil. Fifteen of the 26 DCP holes had what is described as a void at the upper portion of the subgrade. Voids at the upper portion of the subgrade ranged in thickness from 0.1 to 6.4 feet. The most notable voids at the top of the subgrade are in DCP 2-8 (6.4 ft.), DCP 2-15 (3.89 ft.), DCP 1-4 (3.08 ft.), DCP 2-4 (1.07 ft.), DCP 2-3 (0.81 ft.), DCP 1-5 (0.78 ft.), and DCP 2-1 (0.71 ft.). Figure 4-7, Top of Subgrade Topographic Map Based on Dynamic Cone Penetrometer Tests, provides the drill-hole locations and contour of the top of competent (greater than 1 blow per 2 inches) subgrade based on DCP testing. Seventeen voids were identified that exist at some depth within the subgrade. These voids range in thickness from 0.12 ft. to 5.99 ft. The most notable voids in this category are in DCP 2-6 (5.99 ft.), DCP 1-6 (3.79 ft.), and DCP 2-13 (1.94 ft.). The deepest void is a 0.23 ft. void in DCP 2-13 that occurs between 16.06 and 16.29 ft. below the Turbine Building Basement floor (Elevations 973.94 to 973.71).

Eleven of the voids occur at or below the bottom elevation of the pile caps (983.5). These voids range in thickness from 0.15 to 6.54 feet. A summary of the voids encountered during the DCP investigation is presented in Table 4-2.

150

Table 4-2 - Turbine Building Basement Subgrade Investigation DCP Results							
Drill-hole Number	Depth to top of Void from TOS (ft.)	Depth to Bottom of Void from TOS (ft.)	Elevation Top of Void (ft.)	Elevation Bottom of Void (ft.)	Zero blow count /soft zone Thickness (ft.)	Depth to top of Void from Floor (ft.)	Depth to Bottom of Void from Floor (ft.)
1-1	0.00	0.20	987.42	987.22	0.20	2.58	2.78
1-3	0.00	0.10	987.06	986.96	0.10	2.94	3.04
1-4	0.00	3.08	986.58	983.50	3.08	3.42	6.50
1-5	0.00	0.78	987.19	986.40	0.78	2.81	3.60
1-6	6.58	10.38	980.33	976.54	3.79	9.67	13.46
2-1	0.00	0.72	987.44	986.72	0.72	2.56	3.28
2-2	2.54	3.00	985.06	984.60	0.46	4.94	5.40
2-3	0.00	0.81	987.44	986.63	0.81	2.56	3.37
2-3	3.35	3.70	984.09	983.74	0.35	5.91	6.26
2-4	0.00	1.07	987.46	986.39	1.07	2.54	3.61
2-5	0.00	0.33	987.31	986.98	0.33	2.69	3.02
2-6	0.00	0.35	986.92	986.57	0.35	3.08	3.43
2-6	1.32	1.50	985.60	985.42	0.18	4.40	4.58
2-6	2.64	8.63	984.28	978.28	5.99	5.73	11.72
2-7	1.64	1.82	985.57	985.39	0.18	4.43	4.61
2-7	2.96	3.18	984.25	984.03	0.22	5.75	5.97
2-8	0.00	6.54	987.17	980.63	6.54	2.83	9.38
2-8	7.07	7.51	980.10	979.66	0.44	9.90	10.34
2-8	12.26	12.54	974.91	974.63	0.28	15.09	15.38
2-9	1.51	2.25	985.72	984.98	0.74	4.28	5.02
2-10	0.00	0.55	987.27	986.72	0.55	2.73	3.28
2-11	0.73	0.85	986.45	986.34	0.12	3.55	3.66
2-11	2.46	2.64	984.73	984.55	0.18	5.27	5.45
2-12	0.00	0.20	987.44	987.24	0.20	2.56	2.76
2-13	7.27	9.21	980.15	978.21	1.94	9.85	11.79
2-13	13.48	13.71	973.94	973.71	0.23	16.06	16.29
2-15	0.00	3.89	987.27	983.38	3.89	2.73	6.62
2-15	6.99	7.14	980.28	980.13	0.15	9.72	9.87
2-16	0.00	0.69	987.10	986.41	0.69	2.90	3.59
2-17	1.35	1.99	986.09	985.45	0.64	3.91	4.55
2-19	0.00	0.19	987.67	987.48	0.18	2.33	2.52
2-19	2.62	2.79	985.05	984.88	0.17	4.95	5.12
TOS-Top of Subgrade							

4.1.5.3 Discussion/Conclusions

The Turbine Building basement floor drilling and subgrade testing identified both a number of significant voids/soft spots as well as zones of competent soil. Table 4-2 provides the drill-hole number (see Figure 4-3 for drill-hole locations), depth to void and thickness of soft zone (per DCP). The lateral extent and interconnectedness of identified voids can only be inferred from the available data. However, some zones such as the voids encountered in DCP 2-6, DCP 1-4, DCP 1-5 and DCP 2-8 are both significant enough and close enough in lateral distance that we conclude that these voids are part of a connected void system. All of these voids are close to where both the 10-inch and 6-inch drain lines run adjacent to each other and have multiple bends where joints may be more susceptible to cracking or separation in the pipe. In this scenario, significant groundwater inflow into the drainage system is likely. There also are, however, zones where there is little to no evidence of voids or subgrade deterioration such as in testing locations 2-10, 2-11, 2-12, 1-8, 2-14, and 2-18. Overall the data supports the following:

- The Triggering Mechanism of subsurface piping of soil material due to the sump operation and seepage/flow into the drainage system pipes is occurring.
- Voids are significant and interconnected.
- The foundation subgrade is not affected uniformly by this Triggering Mechanism.

Regarding CPFM 3b - Loss of lateral pile support due to subsurface erosion and piping (due to pumping) for the Turbine Building. As discussed in Section 5.8.3.2, the thickness of the void and the potential effects on lateral pile support were considered. The maximum void thickness is 6.54 feet in DCP test location 2-8 and the elevation of the bottom of this void is 980.63 ft. The deepest void in DCP test location 1-6 is 3.79 feet thick with the bottom at elevation 976.54. For the worst case in the collected data, pile support could be lost in limited areas to elevation 976.54. The maximum length of loss of lateral piling support due to a void, calculated from the bottom of the pile cap at elevation 983.5 to the lowest void bottom elevation at 976.54, is 7 feet. There are a total of 10 locations where zero blow count zones exist at elevation greater than three feet below the bottom of slab elevation of approximately 987 (bottom of pile cap elevation 984). Of these 10 zero blow count zones, only five are greater in thickness than 1 foot. The remainder of the drill-hole locations have competent or greater than 1 blow per two inch material to within 3 feet of the bottom of the foundation slab or at the bottom of the pile cap elevation (el. 984).

Based on the available information and without a quantitative analysis we find that the loss of lateral piling support shown by the collected data under the Turbine Building, over the limited areas suggested by the collected data, does not infer that a significant risk of piling failure is present in static conditions due to the presence of the existing voids. Therefore, we have ruled out CPFM 3b for the Turbine Building. It should be noted that the subsurface erosion piping Triggering Mechanism is ongoing and that lateral pile support could be compromised in the future if void thickness and extent continues to increase. Seismic considerations have not been assessed for this report and we do not make any conclusion with respect to the effect of voids on lateral pile support during seismic loading.

The data from the Turbine Building sub-slab investigations cannot be used to rule out CPFM 3b for other pile-supported structures in the vicinity of the Turbine Building, including: Containment Building, Auxiliary Building, Service Building, Circulating Water System, Turbine Building South Switchyard, and the Fuel Oil Storage Tanks and Piping.

Two other CPFMs associated with KDI #1 and Triggering Mechanism #3 have not been ruled out by the Turbine Building sub-slab investigations and have the potential to continue to affect structures other than the Turbine Building. They are:

CPFM 3a – Undermining and settlement of shallow foundation/slab/surfaces (due to pumping)
CPFM 3c – Undermined buried utilities (due to pumping)

Structures potentially affected include: Technical Support Center, Fire Protection System, Raw Water Line, Security BBRE's, Maintenance Shop, Underground Cable Trench (Trenwa), Waste Disposal Piping, Main Underground Cable Bank, Blair Water System, Demineralized Water System, Turbine Building South Switchyard, Fuel Oil Tanks and Piping, PA Paving/Sidewalks/Outdoor Drives, Sanitary Sewer System, and Condensate Storage Tank (buried utilities portion of system). For this Triggering Mechanism to affect these structures, a void would have had to progress beyond the Turbine Building Basement foundation and extend under the Structures listed above. The fact that the flow into the broken pipes has been occurring for many years makes the hypothesis that the voids could have extended beyond the Turbine Building foundation and under the Structures listed above more plausible. The collected data showing that voids were found at the perimeter of the Turbine Building basement between the pile caps at 11 locations also suggests that piping of material from beyond the Turbine Building Basement subgrade may have occurred.

The Triggering Mechanism of subsurface erosion/piping of soil from beneath the Turbine Building basement and perhaps beyond continues as long as the drain system piping remains unrepaired. Voids, soft zones, and associated groundwater and piping flow paths will continue to enlarge and extend out from the drainage and sump system over time unless the flow of water into the sump system is stopped. Therefore, CPFMs 3a, 3b and 3c for the structures listed above cannot be ruled out and remain credible until the following remedial recommendations are implemented to stop the Triggering Mechanism.

4.1.5.4 Recommendations

HDR recommends that OPPD perform remedial work to stop the uncontrolled drainage of the groundwater into the broken Turbine Building basement drainage system piping and fill the voids beneath the basement floor slab. The first priority is to stop the drainage of groundwater into the drainage system as quickly as possible to stop the subgrade erosion process. The quickest and easiest way to stop the flow of groundwater into the sump is to block the drainage system pipes at their termination points into the sump. An alternative to the repair of the existing drainage system is to abandon the existing system entirely, and replace it with an above-structural-floor-slab system. One option to implement this alternative would be to construct a new system that is entirely above basement floor that would utilize pump(s) to remove water from the existing floor drains and the turbine drains. Another option would be to trench cut the 7 inch concrete topping on the structural slab to allow space for installation of new drain pipes. Both these options would facilitate monitoring and access to the system should repairs be necessary.

In addition to drainage system repair, we recommend the voids created by the subsurface erosion/piping caused by the groundwater flow through the broken drainage system pipes be grouted to reestablish the foundation subgrade integrity. This program is for the purpose of preventing further subgrade deterioration that could potentially affect pile lateral support over time and extend beyond the Turbine Building over time. Since the extent of the voids cannot be defined beyond the perimeter of the Turbine Building, we further recommend that the volume of the material it takes to fill the voids be measured to provide a proof of the extent of the voids.

The repair/replacement of the drainage system and filling of the voids to return the foundation soils and subgrade to pre-pipe break condition will allow us to rule out CPFMs 3a, 3b, and 3c for the Structures listed in Section 4.1.3 above. To fill the voids and determine the volume of the voids/zero blow count zones a grouting program should be implemented.

The grouting program design should include:

- Specifications for a grout mix that has the proper rheologic and chemical properties to ensure a balanced, stable mix that will maximize penetration and long-term performance.
- If pipes are abandoned, a monitoring program to establish groundwater conditions without the drain pipe and sump operation should be developed and enacted in order to properly characterize conditions that must be addressed during grouting.
- Specifications for a grout mix that can displace very soft disturbed zones and that can provide long term support for the piles, footing, and slabs.
- Identification of the grout pressure(s) necessary to provide for maximum grout penetration into voids, and soft zones within the subgrade soil material.
- Identification of the maximum grouting pressure allowable to avoid damage to any structures and utilities. Particular attention should be given to the under-slab drain pipes in the event that they are repaired and re-used.
- A plan for real-time, full-time, monitoring and recording of:
  - Grout volumes and pressures under the direction of a qualified engineer at the time of the grouting.
  - Any movement of key Structures during grouting operations.
  - Groundwater elevations outside of the Turbine Building Basement during and after the grouting operation.
- A sequence/logic tree for grout program progression.
- A plan for the drilling of verification holes to include permeability tests to assess the affect of the grouting program on the subgrade soils.
- A grouting acceptance criteria by the Engineer.
- A system to report all of the grouting and monitoring data on a daily basis to the Engineer.
- A final report including all data, results and conclusions developed by the grouting contractor. This should include data on grouting locations, grout takes for each location, verification holes and results, and monitoring and any other data that would support the conclusion that the subsurface voids have been filled.

As mentioned previously, we recommend that OPPD consider abandoning the existing drainage pipes that are in place below the Turbine Building basement floor slab. Attempting to grout the voids after the existing drainage pipes have been repaired will likely damage or even crush the pipes and complicate the grouting process to the detriment of the overall remediation.

In conclusion, this specialized type of grouting operation is necessary both to properly treat the subsurface voids and soft zones and to provide verification/documentation that the program was a success. We recommend the selection of a specialty grouting contractor experienced in performing this type of work. Pre-bid selection criteria should be developed and potential bidders should be pre-qualified based on the selection criteria.

At the time of the writing of this report, it was not certain that a grouting contractor could be found that could implement a program that would yield the data necessary to rule out the remaining CPFMs described above. Discussions with specialty grouting contractors will be scheduled as soon as possible in the future to ascertain if they have the capability to provide the data necessary to rule out the remaining CPFMs.

4.2 Pavement Failure and Sinkhole in Paved Access Area Between Intake Structure and Service Building

Key Distress Indicator #2 is the failure of paving and development of a sinkhole below the roadway paving a few feet west of the Condensate Storage Tank. This roadway is part of a U-shaped area of paved surface that wraps around the northeast, east, and southeast perimeter of the contiguous Power Block buildings (Paved Access Area). The inside of the U-shaped Paved Access Area is defined as the north exterior face of the Maintenance Shop on the north, the east exterior face of the Maintenance Shop and the Service Building on the east, and the south exterior faces of the Turbine and Service Buildings on the south. The outside of the U shape is the south exterior face of the New Warehouse on the north, the Trenwa Cable Trench along the Missouri River on the east, and the north exterior face of the Security Building and the Trenwa Cable Trench from the Security Building west to the end of the pavement on the south, which is generally aligned with the southeast corner of the Turbine Building South Switchyard.

The Paved Access Area overlies a number of structures (buried utilities) between the Power Block and the Intake Structure. The base below this area was excavated to el. 973 ft during construction. Current top-of-paved-surface elevation is approximately 1004.5 ft. Concrete pavement slabs at the surface are underlain by a crushed rock base. This pavement section overlies structural fill down to el. 973 ft with the exception of the area overlying the Circulating Water Tunnels, where fill is placed above the structure, which has a top elevation of 997 ft.

4.2.1 Physical Observations

A number of physical observations made during the facility assessments have been grouped under Key Distress Indicator #2:

- Softened subgrade
- Pavement joint offsets
- Voids under pavement
- Water hydrant failure
- Water seepage at BBRE F-2, MH-5, Intake Structure, and Security Building

4.2.2 Triggering Mechanisms

Seven possible Triggering Mechanisms that might be the root cause of this Key Distress Indicator are as follows:

- Subsurface erosion and piping (due to pumping)
- Subsurface erosion and piping (due to rapid river drawdown)
- Rapid river drawdown, river bank slope failure/spreading

- Soil collapse
- Frost effects
- Hydrostatic lateral loading
- Buoyancy

4.2.2.1 Subsurface Erosion and Piping (Due to Pumping)

Multiple connected seepage paths have the potential to exist in the soil backfill at the site. The paths could be exposed at some locations to the river floodwaters (e.g., a hole in the ground north of the Security building). This potential network of seepage paths could be connected to several pumping sources: the sump in the Turbine Building, Manhole MH-5, and a series of surface pumps inside the perimeter of the Aqua Dam.[1] The dewatering pumps inside the Aqua Dams were operated for an extended period, maintaining a head differential on any potential seepage path networks. Gradient may have been sufficient to begin erosion of surrounding soil.

Unfiltered seepage into the Turbine Building sump continues, so the erosion has the potential to continue until that seepage is stopped. The potential subsurface erosion/piping caused by the Turbine Building sump pumping could extend under the Paved Access Area. Voids could be created under the pavement and along the utility trench walls or pipes. The potential damage includes settlement of pipe or thrust blocks. Settlement can overstress a pipe, can cause a pipe to break, or can cause the displacement of a thrust block, which, in turn, could cause failure of a pipe operating under pressure.

4.2.2.2 Subsurface Erosion and Piping (due to Rapid River Drawdown)

This Triggering Mechanism of subsurface erosion/piping is initiated by river drawdown. Instead of pumping causing a significant groundwater gradient, the groundwater gradient is created by a rapidly receding river level. River level drops faster than pore water pressure in the soil can dissipate. The resulting gradient could be sufficient to begin erosion of the soil along the seepage path.

4.2.2.3 Rapid Drawdown, River Bank Slope Failure/Lateral Spreading

The Triggering Mechanism of slope failure or spreading could occur when the river level drops faster than pore water pressure in the soil can dissipate. The saturated soil is elevated above the dropping river level. The open bank of the river provides no lateral support for the saturated soil and the result is an impending slope failure. If the soil's shear strength is exceeded, the slope will fail along the path of least resistance. Generally slope failures associated with rapid drawdown are relatively localized and shallow in nature.

4.2.2.4 Soil Collapse (first time wetting)

The Triggering Mechanism of soil collapse due to first time wetting occurs when loose soil (spoils with high void ratios and corresponding low dry densities) is saturated for the first time. Saturation of the soils lubricates the soil particles and increases the pore pressure in the soil,

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[1] An Aqua Dam is an engineered water barrier used to contain, divert, and control the flow of water. It consists of two polyethylene liners contained by a single woven geo-tech outer tube. When the two inner tubes are filled with water, the resulting pressure and mass create a stable, non-rolling wall of water (Layfield Environmental Systems, 2008).

loosening the bond between the soil particles. This allows the soil particles to shift into a more compact alignment as the pore water pressure dissipates. The result is a decrease in the soil's void ratio and an increase in dry density. This change in volume is observed as settlement at the ground surface.

4.2.2.5 Frost Effects

The Triggering Mechanism of Frost Effects occurs as soil freezes. Frost effects occur as both frost penetration and uplift, and as frost heave. Completely saturated soils allow frost to penetrate more deeply. Frost penetration and uplift occurs as the water contained in the soil void spaces freezes and expands. Frost heave occurs as ice lenses form and grow from capillary water movement. The change in volume as the water freezes, and as the ice loses form, causes heave at the ground surface.

4.2.2.6 Hydrostatic Lateral Loading

The Triggering Mechanism of Hydrostatic Loading occurs when water levels rise, imposing additional lateral pressure on structures.

4.2.2.7 Buoyancy

The Triggering Mechanism of Buoyancy occurs due to a rise in water or groundwater elevation. Uplift forces occur when the weight of the buried structures is less than the weight of the water or groundwater it displaces. Increased water or groundwater levels increase the buoyancy uplift force on the buried structure.

4.2.3 Structures and CPFMs Associated with Triggering Mechanisms

The Triggering Mechanisms outlined could apply to the following structures and CPFMs:

- Intake Structure
  - CPFM 12a – Rapid Drawdown. Riverbank slope failure and undermining surrounding structures.
  - CPFM 12b – Rapid Drawdown. Lateral spreading.
- Security Building
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3d – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to river drawdown).
  - CPFM 12a and 12b – Rapid Drawdown. River bank slope failure/lateral spreading.
- Security BBREs
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3d – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to river drawdown).
  - CPFM 12a and 12b – Rapid Drawdown. River Bank slope failure/lateral spreading.
  - CPFM14a – Frost Effects.

- Turbine Building South Switchyard
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3b – Subsurface Erosion/Piping. Loss of lateral support for pile foundation (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Condensate Storage Tank
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
  - CPFM 3f – Subsurface Erosion/Piping. Undermined buried utilities (due to river drawdown).
  - CPFM 12a and 12b – Rapid Drawdown. River bank slope failure/lateral spreading.
- Underground Cable Trench (TRENWA)
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
  - CPFM 3d – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to river drawdown).
  - CPFM 3f – Subsurface Erosion/Piping. Undermined buried utilities (due to river drawdown).
  - CPFM 14a – Frost Effects.
- Circulating Water System
  - CPFM 3b – Subsurface Erosion/Piping. Loss of lateral support for pile foundation (due to pumping).
  - CPFM 12a and 12b – Rapid Drawdown. River bank slope failure/lateral spreading.
- Demineralized Water System
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
  - CPFM 3f – Subsurface Erosion/Piping. Undermined buried utilities (due to river drawdown).
- Raw Water Piping
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
  - CPFM 3d – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to river drawdown).
  - CPFM 3f – Subsurface Erosion/Piping. Undermined buried utilities (due to river drawdown).
- Fire Protection System Piping
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
  - CPFM 3d – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to river drawdown).
  - CPFM 3f – Subsurface Erosion/Piping. Undermined buried utilities (due to river drawdown).
- Waste Disposal Piping
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
  - CPFM 3d – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to river drawdown).
  - PFM 3f – Subsurface Erosion/Piping. Undermined buried utilities (due to river drawdown).
  - CPFM 12a and 12b – Rapid Drawdown. River bank slope failure/lateral spreading.

- Fuel Oil Storage Tanks and Piping
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3b – Subsurface Erosion/Piping. Loss of lateral support for pile foundation (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
  - CPFM 3d – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to river drawdown).
  - CPFM 3f – Subsurface Erosion/Piping. Undermined buried utilities (due to river drawdown).
  - CPFM 4c – Hydrostatic Lateral Loading (water loading on structures). Wall failure in flexure.
  - CPFM 4d – Hydrostatic Lateral Loading (water loading on structures). Wall failure in shear.
  - CPFM 4e – Hydrostatic Lateral Loading (water loading on structures). Excess deflection.
  - CPFM 6a – Buoyancy, Uplift Forces on Structures. Fail tension piles.
  - CPFM 6b – Buoyancy, Uplift Forces on Structures. Cracked slabs, loss of structural support.
  - CPFM 6c – Buoyancy, Uplift Forces on Structures. Displaced structure/broken connections.
  - CPFM 12a and 12b – Rapid Drawdown. River bank slope failure/lateral spreading.
- Main Underground Cable Bank, Auxiliary Building to Intake Structure
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
  - CPFM 3d – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to river drawdown).
  - CPFM 3f – Subsurface Erosion/Piping. Undermined buried utilities (due to river drawdown).
  - CPFM 4c – Hydrostatic Lateral Loading (water loading on structures). Wall failure in flexure.
  - CPFM 4d – Hydrostatic Lateral Loading (water loading on structures). Wall failure in shear.
  - CPFM 4e – Hydrostatic Lateral Loading (water loading on structures). Excess deflection.
  - CPFM 6b – Buoyancy, Uplift Forces on Structures. Cracked slabs, loss of structural support.
  - CPFM 6c – Buoyancy, Uplift Forces on Structures. Displaced structure/broken connections.
  - CPFM 12a and 12b – Rapid Drawdown. River bank slope failure/lateral spreading.
- Blair Water System
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Camera Towers and High Mast Lighting
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3d – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to river drawdown).
  - CPFM 12a and 12b – Rapid Drawdown. River bank slope failure/lateral spreading.
- Service Building (Priority 2 Structure)
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Maintenance Shop (Priority 2 Structure)
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).

4.2.4 Assessment Methods and Procedures

Assessments were made by walking the Paved Access Area and observing surface features of the system (manholes) and the ground surface. The surface assessment included using a 4-ft-long, 0.5-in.-diameter, steel-tipped fiberglass T-handle soil probe to hand probe the adjacent ground surface along the utility alignments and areas to determine relative soil strength. The assessment focused on identifying conditions indicative of potential flood-related impacts on or damage to the utility as follows:

- Ground surface conditions overlying and immediately adjacent to the Paved Access Area
- Soft ground surface areas as determined by probing
- Water accumulations and flows in subsurface system components (manholes and concrete cable encasement pipes)
- Damage to at-grade or above-grade system features
- Variance from normal installation conditions including settled, tilted, or heaved system features and equipment
- Operation of the system and appurtenant equipment (i.e., is the system operational?)

Additional investigations were performed to further characterize the subsurface at the facility including areas where conditions indicative of potential flood-related impacts or damage were observed. These included the following non-invasive geophysical and invasive geotechnical investigations. Results of these tests are described in Section 4.4 of this Assessment.

- GPR
- Seismic surveys (seismic refraction and refraction micro-tremor)
- Geotechnical investigations including test borings with field tests (SPT and cone penetration test [CPT]) and laboratory tests. Note that OPPD required vacuum excavation for the first 10 ft of proposed test holes to avoid utility conflicts. Therefore, test reports will not show soil conditions in the upper 10 ft of test boring logs.
- Paved areas were evaluated with GPR and dynamic deflection methods (i.e., drop weight deflectometer).

4.2.5 Recommended Actions

The following actions are recommended for this Key Distress Indicator/Triggering Mechanism.

4.2.5.1 Detailed Forensic Investigations

Review of GPR and seismic refraction surveys indicates zones of relatively lower density material. In addition, drop weight deflectometer tests reveal additional potentially degraded zones. A plan and profile view of the Paved Access Area should be developed showing the suspected zones of lower density material. These zones should be geo-referenced so that they can be located and marked on the ground surface.

Selected sections of pavement will be removed from the paved access area between the Intake Structure and Service Building. All lower density zones identified within 5 ft of the ground surface by the aforementioned assessment methods should be investigated with test pits. Test pits should be carefully excavated with a backhoe or hand excavation to the extent possible in order to prevent damage to any existing utilities. Soil samples should be collected and tested to establish material characteristics such as Atterberg Limits, particle gradation, and moisture-density relationships. The bottom of each trench should be probed with a rod to establish the general limit of the soft material.

Excavation to greater depths could be considered if OPPD and HDR are confident that no potential damage to any utilities exists.

All excavated trenches will be backfilled according to existing pavement subgrade specifications.

The rationale for this trenching and testing program is to quantify the condition of the subgrade of the paved access area between the Intake Structure and Service Building to the extent feasible and to provide analysis and recommendations for the extent and method of necessary subgrade repair.

4.2.5.2 Physical Modifications

Damaged pavement in the paved access area should be removed. After removal, the subgrade should be inspected for voids and soft soils. The subgrade should then be stabilized, and new pavement should be installed. The extent of physical modifications required for the paved access area between the Intake Structure and the Service Building cannot be defined at this point. Based on observations, it is reasonable to assume that some improvements to the subgrade will be necessary. HDR will define the nature and extent of required physical modifications based on the findings of the Detailed Forensics Investigation.

4.2.5.3 Continued Monitoring Program

Continued monitoring of the paved access area between the Intake Structure and Service Building is recommended to include visual inspections of pavement slabs, structures, and surrounding grades; also recommended is the continuation of the elevation surveys of the previously identified targets on the structure and surrounding site. The purpose is to monitor for signs of structure distress and movement or changes in soil conditions around the structures listed in Section 4.2.3.

The results of this monitoring will be used to increase the confidence in the assessment results. Elevation surveys and visual inspections should be performed weekly for 4 weeks and biweekly until December 31, 2011. If any new distress indicators are observed between inspection intervals by OPPD staff, the proper personnel should be notified immediately to determine if an immediate inspection or assessment should be conducted.

4.2.6 KDI #2 Forensic Investigation

Forensic investigation to address KDI #2 consisted of field observation and testing of subsurface soils exposed through excavation of trenches and removal of concrete pavement at selected locations, test borings and field and laboratory tests, and evaluation of inclinometer and survey monitoring data. KDI #2 consists of a number individual distress indicators observed within the PAA including softened subgrade, pavement settlement, a void beneath the pavement in one location, water hydrant failure, and water seepage at BBRE-F2, MH 5, and the Intake Structure and Security Building.

Possible Triggering Mechanisms identified for KDI #2 include:

- Subsurface erosion and piping (due to pumping)
- Subsurface erosion and piping (due to rapid river drawdown)

These Triggering Mechanisms and related Structures/CPFMs are discussed in detail in Sections 4.2.2 and 4.2.3. Conclusions related to these are discussed below in Section 4.2.6.3.

The purpose of this investigation was to determine the presence and extents of potential voids and soft zones in the subsurface, lateral or vertical movement in the subsurface, and evaluate which of the Triggering Mechanisms and associated CPFMs identified for KDI #2, if any, appear to be responsible for the observed distresses.

4.2.6.1 Scope of Work

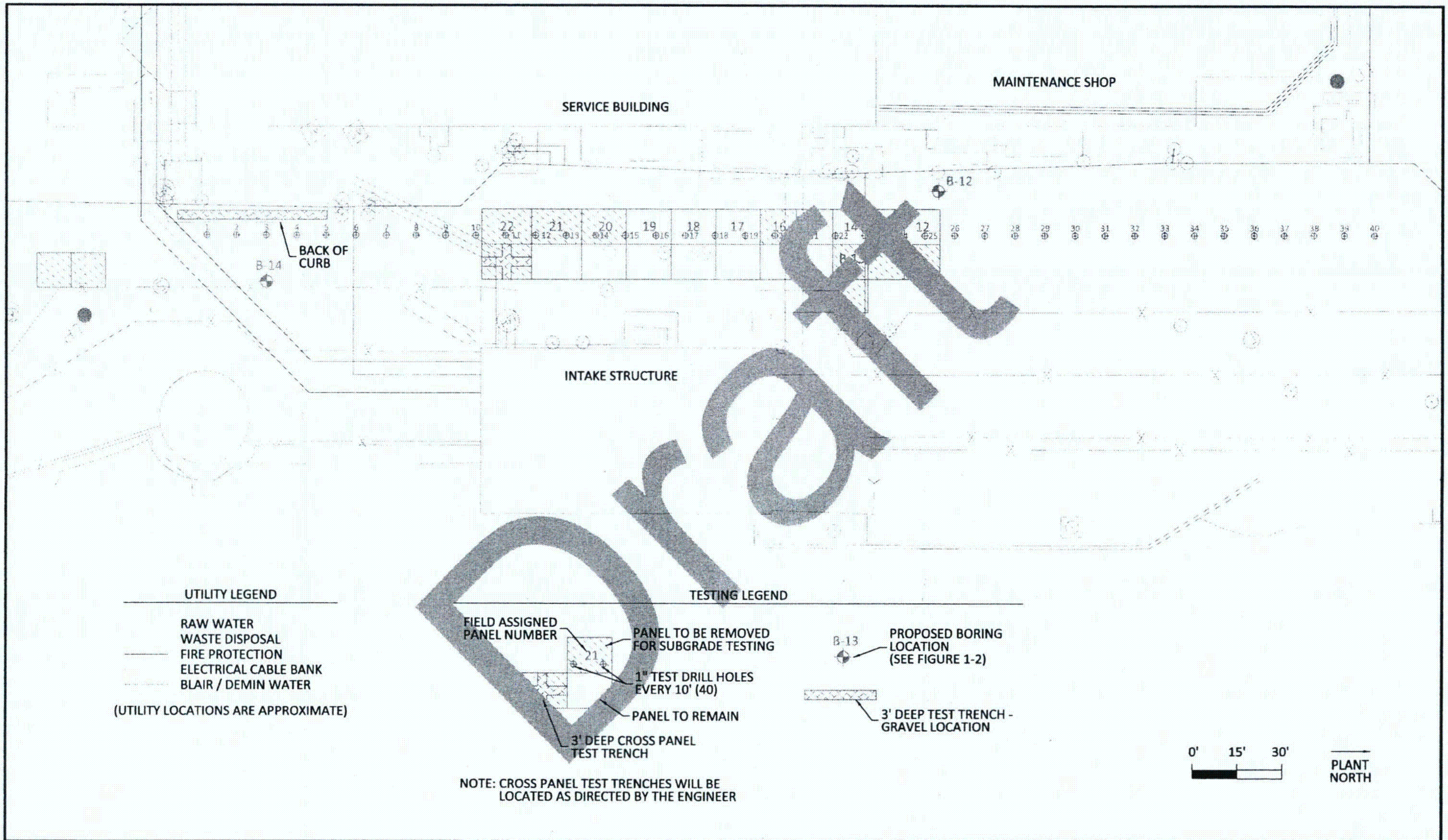
Trench excavations and concrete removals and associated field observation and testing, Standard Penetration Test (SPT) borings and field tests, and inclinometer and survey monitoring were performed between August 2011 and January 2011. These activities are described below.

4.2.6.1.1 Excavation and Subgrade Testing

Trenching and concrete removals and associated field observation and testing were completed November 28 through December 13, 2011. Trenches were excavated and the exposed excavation floor soils tested in two locations, both within the gravel surfaced area located adjacent to, and extending to the north of, Manhole MH-5. Concrete was removed, generally in full-panel sections, and subgrade testing performed at four locations across the main corridor of the PAA identified as, from south to north, the Void Panels Area, South Panels Area, Panel 16 Area, and North Panels Area. These test areas are illustrated in Figure 4-9, Pavement Excavation and Subgrade Testing Areas.

Evaluation of the trenches and exposed pavement subgrade included observation of soil conditions and in-situ field testing. Observations of exposed subgrade were made by HDR and Thiele. Subgrade field testing was performed solely by Thiele as directed by HDR.

Observation included continual visual observations of the subsurface soils and pavement subgrade that were exposed as excavations and concrete pavement removals progressed and following their completion. HDR also evaluated the exposed materials using a pointed, metal tipped T-handle probe (commonly referred to as a foundation probe) where the probe was pushed into the exposed surface by hand to qualitatively evaluate relative consistency/firmness and depth of detected soft areas.



Omaha Public Power District

Pavement Excavation and Subgrade Testing Areas
Fort Calhoun Station

Paved Access Area Field Investigation Program

HDR

DATE
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FIGURE
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In-situ field tests on exposed trench floor soil and pavement subgrade consisted of static cone penetrometer (SCP) tests. Thiele used a Humboldt Model H-4210A Portable Static Cone Penetrometer to perform the SCP tests. This device consists of a direct-read penetrometer that measures the amount of penetration resistance as the device is pushed into soil materials. The cone penetrometer was advanced into the soil by hand in continuous six-inch vertical intervals until refusal or the maximum vertical reach of the device (3.0 feet) was reached, whichever occurred first. Resistance readings were observed and recorded for each 6-inch interval. Measurements of the depth into the subgrade at each location tested were made using gradation markings on the cone penetrometer shaft.

SCP tests were performed at a frequency of about 1 test per square yard of exposed soil (trench excavation floor or exposed concrete pavement subgrade) and were generally completed at 3-foot horizontal intervals. Exceptions to this included two areas of exposed concrete pavement subgrade, one made inaccessible by a soil stockpile and one area that was not part of the originally planned investigation, as described below.

The trench excavations were identified as Trench TE-1 and Trench TE-2. Both trenches were excavated to depths of about three feet below existing ground surface with a Bobcat 325 track excavator using a 2-foot wide bucket. Trench TE-1 measured about 2 feet wide by 30 feet long. Trench TE-2 measured about 2 feet wide by 5 feet long. Concrete was encountered during excavation of Trench T-2 at the north floor of the excavation at a depth of about 3 feet below existing ground surface at which time machine excavation was halted. Further hand excavation exposed the top and southeast side of the Main Underground Cable Bank.

The Void Panels Area included removal of one complete concrete pavement panel and a small (about 3-foot) diagonal cut portion of the adjoining panel to the north (at the small void) located just north of the Security Building, adjacent to the inner-most security fence, west-southwest of the Condensate Storage Tank. This area was investigated to address a subgrade void below a small area (about one foot across) of broken concrete at the expansion joint between two concrete panels. The area of concrete removal and exposed subgrade testing measured about 12 feet by 12 feet (about 144 square feet or 16 square yards). A total of 16 SCP tests were completed in the subgrade at the Void Panels Area.

Activities in the South Panels Area included removal of 7 complete concrete pavement panels and was located along the east side of the PAA main corridor beginning just west of the southwest corner of the Intake Structure. It included the 4 panels originally planned for removal and subgrade investigation, plus 3 additional panels removed at the direction of OPPD. This area was investigated to address possible piping and voids below the concrete pavement or along near-surface utilities and structures. The area of exposed subgrade and testing extended north to south for about 60 feet, with the southern roughly 47 feet (of the 60 feet - 6 pavement panels) measuring about 26 feet east to west and the northern roughly 13 feet (of the 60 feet - one pavement panel) measuring about 14.5 feet east to west (roughly 1,410 square feet or 156 square yards). A total of 112 SCP tests were completed in the subgrade at the South Panels Area. Prior to testing, the subgrade exposed in the northeastern-most portion of the South Panels Area (not part of the originally planned investigation but where concrete was removed at OPPD's direction) was covered by a stockpile of RCC fill and was not SCP tested.

The Panel 16 Area included removal of one concrete pavement panel (field marked by OPPD as Panel 16) located in the central portion of PAA main corridor west of the rollup door to the Intake Structure. This area was investigated to address possible piping and voids below the concrete pavement and/or near-surface utilities and structures. The area of exposed subgrade and testing measured about 4.5 feet north to south and 12 feet east to west (roughly 54 square feet or 6 square yards). A total of 6 SCP tests were completed in the subgrade at the Panel 16 Area.

The North Panels Area included removal of 7 complete panels and one diagonally cut half-panel and was located along the east side of the PAA main corridor beginning just northwest of the northwest corner of the Intake Structure and included 3 of the 4 panels originally planned for removal and subgrade investigation, plus 4.5 additional panels removed at the direction of OPPD. The southeastern-most pavement panel of the originally planned investigation area panels was not removed at the request of OPPD as it was in very good condition and to avoid possible impacts/damage to the immediately adjacent security fence and a fire hydrant. This area was investigated to address possible piping and voids below the concrete pavement and/or near-surface utilities and structures. The area of exposed subgrade and testing extended north for about 89 feet, with the northern-most 15 feet including a diagonal cut at the northeast corner and pan-handle feature extending about 12 to 13 feet to the west. The total area of the North Panels Area was roughly 1,322 square feet (147 square yards) and included about 466 square feet (52 square yards) of originally planned investigation area and 856 square feet (95 square yards) of additional area exposed at the direction of OPPD. A total of 49 SCP tests were completed in the subgrade of the originally planned investigation area with 18 tests completed in the additional exposed subgrade. A relatively reduced testing frequency was performed in the additional area due to time constraints of impending rain and to allow for full-frequency testing in the originally planned investigation area.

Geotechnology, Inc. Seismic Analysis

Geotechnology, Inc. (GTI) conducted seismic evaluations along 5 lines utilizing two different methods of analysis, Refraction and Refraction Microtremor (ReMi). The seismic investigation lines are shown on Plate 3 of the GTI Report dated October 24, 2011 and is presented in Attachment 6. The following is taken from the GTI Report.

"3.1 Seismic Methods

Refraction. The seismic refraction method involves generating compressional seismic waves (p-waves) at the ground surface using an impact source. The seismic waves travel from the source through the subsurface along a variety of paths including refracting along interfaces between soil and rock layers having different seismic velocities. The seismic waves return to the ground surface where they are recorded at various distances from the source using geophones and a seismograph. Seismic velocity calculations are made by analyzing the differences in elapsed time from the source to each geophone. The resulting profile is a representation of p-wave velocities of the soil and bedrock directly beneath the survey line.

Refraction Microtremor (ReMi). The ReMi method is used to develop shear wave velocity profiles. ReMi surveys are conducted by passively recording background surface waves (microtremors) that are generated by passing vehicles, equipment, airplanes, etc. The surface seismic energy produced by the noise sources travels across the ground surface and is received by geophones placed in a linear array. The seismic energy detected at the geophones is recorded using a seismograph and is transformed into a phase velocity spectrum for analysis. Shear wave velocity profiles are constructed by analyzing surface wave phase velocities and frequencies, and performing inversion modeling.

3.4 Seismic Results. The seismic data were interpreted by comparing the velocity profiles to nearby Borings B-4, B-7, -8, and -9, which were used to establish the types of geologic materials corresponding to the profiled velocities. The stratigraphy at the site is generally comprised of approximately 70 feet of alluvial deposits over limestone bedrock. Weathered shale bedrock was observed in a boring immediately north of the subject survey area. The alluvial deposits are comprised of alternating layers of loose and dense silty sand to sand with silt and occasional layers of clay up to 14 feet thick. Sand and clay each exhibit a wide range of velocities depending on a number of physical parameters such as moisture content, porosity, sorting, and particle packing. Based on the seismic refraction data, alluvium at the subject site exhibits velocities ranging between approximately 1,500 ft/s within the top 20 feet of material and increasing with depth to approximately 5,000 ft/s near the top of bedrock. Published P-wave velocities for sands range between 1,300 and 6,500 ft/s and clays may range between approximately 3,500 and 8,200 ft/s. Top of bedrock was interpreted to generally coincide with the 5,000 foot/second (ft/s) contour on the refraction data as shown on Plates 10 through 13. Top of bedrock undulated across the site. The shallowest bedrock imaged appeared to be at a depth of approximately 56 feet at the east end of Line 5 and the deepest bedrock imaged appeared to be at a depth of approximately 78 feet at the west end of Line 5.

The circulation structure located between the main plant building and the Missouri River was not imaged in Lines 2 and 3. The data collected along these lines exhibited significant noise from facility activities and exhibited high-velocity shallow energy from the surface pavement, which masked our ability to pick the arrivals related to the shallow circulation structure.

Zones of low velocity were observed in the refraction and ReMi data above and below the top of bedrock as indicated on Plates 9 through 18. These low velocity zones indicate locations at which material is softer and/or less dense and through which the seismic wave travels slower compared to surrounding material. These velocity contours are gradational and illustrate velocity changes between extreme values. These values do not necessarily represent the actual seismic velocities, but rather, illustrate the general trends of velocity changes across the profiles and the general locations and relative differences of the extreme high and low velocities.

Low velocity features within limestone bedrock could be due to the presence of:

- karst features such as voids, clay or water filled cavities or solution-widened joints/fractures.
- zones of weathered or otherwise weak rock compared to surrounding more competent rock.

Low velocity features within the alluvium could be related to:

- zones of loose sand as observed in nearby borings.
- voids, if sufficient overlying cohesive material is present for bridging."

SPT borings were used to ground-truth the findings of the GTI Seismic Investigation as described in the following paragraphs.

4.2.6.1.2 SPT Borings

Thiele completed six SPT borings (identified as Borings B-10 through B-15, inclusive) November 9 through 17, 2011. The location of the SPT borings and their relationship to the GTI Seismic investigation lines is presented in Figure 4-10. These 6 borings were drilled to ground-truth subsurface anomalies identified as "low velocity features" in the GTI Report One of the SPT borings, B-10, was intended to be a baseline boring and was drilled in an area that did not evidence low velocity features. The other 5 borings were drilled at locations of reported low velocity features. These borings were drilled to bedrock and were continuously sampled so that the low velocity features could be evaluated using SPT test result data. Of these 6 borings, 3 were completed in the PAA (Borings B-12, B-13, and B-14). Boring logs and test results from Borings B-12, B-13, and B-14 were evaluated to address KDI#2.

The borings were continuously sampled from 10 feet below existing ground surface to the maximum depth investigated. The upper-most 10 feet of soil at each boring was hydro-excavated to clear possible underground utilities. Continuous split-spoon SPTs (ASTM D 1586-08a) were performed and soil samples collected during drilling except occasionally where undisturbed Shelby tube samples were collected by direct-push. Where Shelby tube samples were collected, laboratory dry density test results were used for our evaluation. All borings were advanced to auger refusal and terminated on the top of the limestone bedrock formation underlying the site.

A summary of the test borings and seismic anomalies addressed by each is as follows:

- Boring B-12 – intercepted a single anomaly reported as existing from about 32 to 58 feet below ground surface (refer to GTI report, Plate 11).
- Boring B-13 – intercepted two reported anomalies, one existing from about 3 to 20 feet below ground surface and one existing from about 41 to 70 feet below ground surface (refer to GTI report, Plate 9).
- Boring B-14 – intercepted two reported anomalies, one existing from about 6 to 28 feet below ground surface and one existing from about 38 to 53 feet below ground surface (refer to GTI report, Plate 9).

4.2.6.1.3 Inclinometer Monitoring

Thiele performed weekly monitoring of inclinometers (installed into bedrock for this assessment) which began in late November, 2011 and will run through late January, 2012. A total of 5 inclinometers (Inclinometers I-1 through I-5, inclusive) were installed and monitored to evaluate if any lateral movement was occurring at the site related to the 2011 flood. Monitoring results from the inclinometers were reviewed for this KDI #2 forensic investigation to evaluate movement in the PAA possibly related to KDI #2.

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GTI Siesmic Investigation Lines and SPT Boring Locations
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FIGURE	4-10

4.2.6.1.4 Survey Monitoring

LRA provided weekly survey monitoring under contract to OPPD from late August through late December, 2011. 264 survey points associated with 40 structures and site features across the FCS are included in this weekly monitoring. Of these, 49 survey points associated with 9 structures/site features were reviewed for this KDI #2 forensic investigation to evaluate movement in the PAA Structures. These Structures included the Auxiliary Building, BBRE-F2, Condensate Storage Tank, Intake Structure, Maintenance Shop, Security Building, Service building, Turbine Building South Switchyard, and MH-25.

4.2.6.2 Results

Forensic investigation results including field observations, SCP test results, SPT test results, and inclinometer and survey monitoring results are summarized below. Test reports by Thiele and LRA survey monitoring results are included in Attachment 6.

4.2.6.2.1 Excavation and Subgrade Testing

No piping voids or ground subsidence were identified through visual observation, T-handle probing, or SCP tests in any of the locations exposed through trench excavations or concrete pavement removals. Field SCP testing indicated that stiff to very stiff clayey silt to silty clay fill soils were generally encountered in the upper 3 feet below the ground surface or pavement. Occasionally, soft to medium stiff soils were encountered at the 3-foot depth. Some very soft to soft soil was encountered and was generally limited to the upper-most 6 to 12 inches and appeared associated with relatively high moisture content soils (very moist to wet) associated with concrete pavement expansion joints (joints between adjacent panels) and surface run-on from adjacent pavements related to precipitation (rain and snow) that occurred during the work.

The top and southeast side of the monolithic concrete Main Underground Cable Bank were exposed and observed in Trench T-2. The top of the structural concrete Circulation Water Tunnel structure was exposed at a few locations by hand excavations completed in the subgrade exposed in the South Panels Area. The fill exposed at both of these concrete features was compact fine-grained cohesive material and showed no evidence of piping erosion or excessive moisture.

4.2.6.2.2 SPT Borings

Material encountered in the subsurface at Borings B-12, B-13, and B-14 generally consisted of alluvium including poorly graded, fine- to coarse-grained sand (SP) and to a lesser extent silty sand (SP-SM) and clayey sands (SC). Silt and lean clay zones were encountered in Borings B-13 and B-14 in the upper 10 to 20 feet; these soils were logged as fill and documented as such in various historical geotechnical reports and as-built drawings provided by OPPD.

No voids or very soft/very loose conditions that might be indicative of piping or related material loss or movement were identified through drilling and continuous sampling of the test borings. N-values (uncorrected) indicated that the encountered alluvium ranges from loose to medium dense and that soil conditions were similar between anomalous zones (low velocity features reported by GTI) and non-anomalous zones. The reported low velocity zones are attributed to the inherent variability in the relative density of the granular alluvium that underlies the site. SPT results were compared to similar data from numerous other geotechnical investigations that have been conducted on the FCS site in previous years and for this assessment at other locations across the site. This comparison did not identify apparent differences from soils encountered at other on-site test boring locations nor did it identify changes in soil relative density following the 2011 flood.

4.2.6.2.3 Inclinometer Monitoring

Data from inclinometers to date, compared to the original baseline measurements, have not exceeded the accuracy range of the inclinometers. Therefore, deformation at the monitored locations since the installation of the inclinometers has not occurred.

4.2.6.2.4 Survey Monitoring

Survey data points to date (in the PAA) compared to the original baseline surveys have not exceeded the accuracy range of the surveying equipment. Therefore, deformation at the monitored locations, since the survey baseline was shot, has not occurred.

4.2.6.3 Discussion and Conclusions

Forensic investigation as described above was performed where observed pavement distress was most prominent, at locations coincident with shallow underground structures and utilities, and where recent seismic surveys identified low velocity features (locations where potential for degradation related to the Triggering Mechanisms and CPFMs associated with KDI #2 was identified).

Excavation and subgrade testing identified no evidence of piping erosion, voids, or subsidence of site fills. Field SCP testing of the exposed subgrade indicated that stiff to very stiff soils were generally encountered in the upper 3 feet below the ground surface or pavement. Based on the observations made and tests results obtained, the fill soils in the locations exposed and tested are compact, cohesive soils that are not susceptible to piping erosion. SPT borings did not identify voids or very soft/very loose conditions that might indicate piping or related material loss nor did they identify changes in soil relative density following the 2011 flood. Inclinometer and survey monitoring indicates that movement of on-site subsurface soils or structures has not occurred.

Possible Triggering Mechanisms and related CPFMs identified for KDI #2 and the PAA include:

- Subsurface Erosion and Piping (due to pumping), CPFMs 3a, 3b, and 3c.
- Subsurface Erosion and Piping (due to rapid river drawdown), CPFMs 3d, 3e, and 3f.

Based on the observations and test results, the individual distress indicators that comprise KDI #2 are not attributed to the possible Triggering Mechanisms identified for KDI #2: Subsurface Erosion and Piping (due to pumping); and, Subsurface Erosion and Piping (due to rapid river drawdown).

Our investigation for KDI #2 also indicates that the Triggering Mechanism of Subsurface Erosion and Piping (due to rapid river drawdown) was not initiated by the 2011 flood and that the CPFMs related to this Triggering Mechanism, including CPFM 3d, 3e, and 3f, are not credible.

However, the Triggering Mechanism of Subsurface Erosion and Piping (due to pumping) and the CPFMs related to this Triggering Mechanism, including CPFM 3a, 3b, and 3c cannot be ruled out for all structures associated with the PAA. Even though this Triggering Mechanism does not appear to have caused the distresses observed in the PAA, their root cause (damaged Turbine Building sub-floor drain pipes and sump pumping) as identified by investigations in the Turbine Building basement continues. A number of other Priority 1 and Priority 2 structures have been assigned CPFMs that are related to this remaining credible Triggering Mechanism and its related CPFMs. These other structures differ from KDI #2 and the PAA in that no strong evidence of distress has been identified or documented through assessment observations or ongoing survey monitoring.

Priority 1 Structures in this category include:

- Security BBREs
- Turbine Building South Switchyard
- Condensate Storage Tank
- Underground (TRENWA) Cable Trench
- Circulation Water System
- Demineralized Water System (line)
- Raw Water Piping
- Fire Protection System Piping
- Waste Disposal Piping
- Fuel Oil Storage Tanks and Piping
- Main Underground Cable Bank, Auxiliary Building to Intake Structure
- Blair Water System
- ~~River Bank~~

Priority 2 Structures in this category include:

- Service Building
- Sanitary Sewer System

The potential for impact to the above Priority 1 and Priority 2 Structures from the Triggering Mechanism of Subsurface Erosion and Piping (due to pumping) exists and the CPFMs related to this Triggering Mechanism remain credible until the recommendations related to KDI #1 as presented below are implemented and completed. Continued monitoring of the above structures will be required after these recommendations are implemented and completed to evaluate if the recommended actions were effective and the CPFMs are therefore no longer deemed credible.

However, it can be concluded that the Subsurface Erosion/Piping Triggering Mechanism (due to pumping) most-likely did not extend outside the perimeter of the Seismic investigation lines taken around the power block. This conclusion supports the ruling out of the Subsurface Erosion/Piping (due to pumping) CPFMs associated with this Triggering Mechanism for the following Structures:

- Security Building
- Intake Structure
- River Bank

4.2.6.4 Recommendations

The results of this KDI #2 forensic investigation have ruled out potential Triggering Mechanisms and associated CPFMs that could have been the cause of the observed distress. However, it could not be used to entirely rule out CPFMs associated with KDI #1, which is associated with the uncontrolled drainage of the groundwater into the broken Turbine Building basement drainage system piping. These CPFMs will only be ruled out when the physical modifications presented for KDI #1, as presented in Section 4.1 of this Assessment Report, are implemented.

4.3 Column Settlement in Maintenance Shop

Key Distress Indicator #3 is the settlement of column TE-15 in the Maintenance Shop. The column is on the first floor of the Maintenance Shop outside the men's restroom adjacent to the northern side of the Turbine Building. OPPD staff has indicated that the column had begun settling prior to the 2011 Flood, and that the settlement had increased during the flood. As of October 7, 2011, the column has settled 2.2 in. In addition to the settled column, there are cracks in the wall nearest the beam adjacent to the men's restroom, and the doors on the restroom no longer operate properly.

4.3.1 Physical Observations

A number of physical observations made during the facility assessments have been grouped under this Key Distress Indicator:

- Significant settlement of a building column (2.2 in.)
- Significant settlement of floor slab
- Cracking of masonry partition walls in the southwest corner of this building immediately adjacent to the Turbine Building

4.3.2 Triggering Mechanisms

Two possible triggering mechanisms that might be the root cause of this Key Distress Indicator are discussed as follows.

4.3.2.1 Subsurface Erosion and Piping (Due to Pumping)

Multiple connected seepage paths have the potential to exist in the soil backfill at the site. This potential network of seepage paths could be connected to several pumping sources: the sump in the Turbine Building, Manhole MH-5, and a series of surface pumps inside the perimeter of the Aqua Dam. The dewatering pumps inside the Aqua Dams were operated for an extended period, maintaining a head differential on any potential seepage path networks. Gradient might have been sufficient to begin erosion of surrounding soil.

Unfiltered seepage into the Turbine Building sump continues, so the erosion has the potential to continue until that seepage is stopped. The potential subsurface erosion/piping caused by the Turbine Building sump pumping could extend underneath the Maintenance Shop.

4.3.2.2 Soil Collapse (first time wetting)

The most recent flood elevation prior to the 2011 flood was 1003.3 ft, which occurred in 1993. The maximum flood elevation in 2011 was approximately 1006.9 ft. The foundation of the maintenance shop has footings at el. 1000.5 ft and subgrade below the flooring slab of approximately el. 1006 ft. Therefore, it is possible that up to 3 ft of soil were saturated for the first time as a result of the 2011 flood. This alone could not cause settlement of the foundation footings due to first time wetting because the footing elevation of 1000.5 ft had likely experienced first time wetting in 1993.

4.3.3 Structures and CPFMs Associated with Triggering Mechanisms

The Triggering Mechanisms outlined could apply to the following structures and CPFMs:

- Security Building
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Security BBREs
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Turbine Building South Switchyard
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
  - CPFM 7a through 7c – Soil Collapse (first time wetting). Cracked slab, differential settlement of shallow foundation, loss of structural support; displaced structure/broken connections; and general site settlement.
- Condensate Storage Tank
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Underground Cable Trench
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Demineralized Water System
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Raw Water Piping
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Fire Protection System Piping
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Waste Disposal Piping
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).

- Fuel Oil Storage Tanks and Piping
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Main Underground Cable Bank, Auxiliary Building to Intake Structure
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Blair Water System
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
  - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Camera Towers and High Mast Lighting
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Service Building
  - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).

4.3.4 Assessment Methods and Procedures

Initial assessments were made by OPPD staff and are described in OPPD reports.

An additional investigation was performed on August 2, 2001, to further characterize the subsurface at the areas where conditions indicative of potential flood-related impacts or damage were observed. A subsurface survey using GPR was performed by Ground Penetrating Radar Systems, Inc. (GPRS). The report is titled "Ground Penetrating Radar (GPR) survey to locate sub surface voids at the Ft. Calhoun Nuclear Facility in Blair, NE." The GPR survey identified potential voids in the soil beneath the column and along the length of the corridor from 8 to 12 in. below the surface. The voids are referred to as "small in thickness in most areas" and slightly thicker nearest the settled column.

4.3.5 Previous Investigations and Baseline Information

Prior to construction of the Maintenance Shop addition, Geotechnical Services, Inc., performed an investigation titled "Report of Subsoil Investigation for Proposed Maintenance Shop Addition" in 1977. Four borings were completed to assess soil conditions. The borings recorded 7 to 9.5 ft of fill material consisting of clayey silt on the south side of the proposed structure area and fine sand on the north side of the proposed structure area. SPT N-values of the fill material range from 9 to 20. Elevations of the borings were not recorded.

Maintenance Shop drawings indicate that the floor elevation is 1007.5 ft, and the elevation of the bottom of the foundation footings is 1000.5 ft. Therefore, based on the depth of fill material below existing grade established in the previously mentioned report, the foundation footings are placed on fill material.

4.3.6 Recommended Actions

The following actions are recommended for this Key Distress Indicator/Triggering Mechanism:

4.3.6.1 Detailed Forensic Investigations

Review of GPR and seismic refraction surveys reveals voids or zones of relatively lower-density material. One-in.-diameter borings through the floor slab should be drilled at the locations where GPR surveys were conducted. A miniature cone designed to record tip resistance should be pushed into the foundation soil material as deep as possible beneath the floor slab in order to record tip resistance. Depths of voids and soft zones encountered will be determined and documented. In addition to the use of a miniature cone, a waterproof bore scope with lighting should be available to investigate further and determine the extent of any voids encountered.

Upon completion, the floor holes would be refilled with non-shrink grout with a minimum 28-day compressive strength of 2000 psi. Repair of holes shall meet OPPD criteria and be approved by OPPD staff.

Upon completion of the proposed drilling and inspection work, OPPD and HDR staff will discuss the necessity and location for additional drilling locations to further define the extent of any voids encountered. Approval by OPPD staff will be required prior to beginning any additional drilling.

The results of this subsurface investigation would be used to determine the existence of voids and low density zones that could be related to the settled column.

4.3.6.2 Physical Modifications

Once the geotechnical evaluation is complete, an engineered design for foundation restoration should be developed. Possible remedial efforts include foundation jacking and underpinning. Ground improvement can include measures such as compaction grouting to increase the density of the subsurface soils.

4.3.6.3 Continued Monitoring Program

Continued monitoring in the Maintenance Shop is recommended to include visual inspections of the area where observed settlement has occurred; also recommended is a continuation of the elevation surveys of previously identified targets on the structure and surrounding site prior to remediation of the Maintenance Shop foundation and structure.

The results of this monitoring would be used to increase the confidence in the assessment results. Elevation surveys and visual inspections should be performed weekly until remediation is complete. Once remediation is complete, specific survey monitoring points should be installed in the remediated area of the Maintenance Shop. These points should be monitored weekly for 2 months after remediation, then once every 3 months for a period of 1 year in order to assess the overall effectiveness of the repair.

4.3.7 KDI #3 Forensic Investigation

Forensic investigation consisting of concrete floor slab drilling and field and laboratory subgrade testing was completed in the Maintenance Shop to evaluate subsurface conditions near Key Distress Indicator (KDI) #3. This Key Distress Indicator consists of differential settlement of Building Column MG-15, presumed differential settlement of the nearby floor slab, and cracked nearby masonry partition walls. These building distresses were observed at the southwest corner of the building immediately adjacent to the north side of the Turbine Building during facility assessments.

Possible Triggering Mechanisms identified for KDI #3 include:

- Subsurface Erosion and Piping (due to pumping); and
- Soil Collapse (due to first time wetting).

These Triggering Mechanisms and related Structures/CPFMs are discussed in detail in Sections 4.3.2 and 4.3.3. Conclusions related to these are discussed below in Section 4.3.7.3.

The purpose of this investigation was to determine the presence and extents of potential voids and soft zones beneath the floor slab and evaluate which of the Triggering Mechanisms identified for KDI #3 are responsible for the observed building distresses.

4.3.7.1 Scope of Work

Forensic investigation of the Maintenance Shop was conducted from November 9 through December 2, 2011. A total of 24 floor slab locations were drilled and the underlying subgrade evaluated by the investigation. This included 1-inch diameter holes at 16 locations as originally planned (drill-holes 1-1 through 1-16). An additional 4, 1-inch diameter holes were drilled and investigated to the east of the original investigation area (drill-holes EW-1 through EW-4) and 2 to the north of the original investigation area (drill-holes NS-1 and NS-2). Shelby tube sampling in 2 test borings was also added to the original scope of work.

Drilling of the concrete for the 22, 1-inch diameter holes was accomplished by Lueder Construction Company under contract to OPPD using a hammer drill. All of the 1-inch diameter drill-holes were protected immediately following drilling and before and after subgrade evaluations using temporary plastic caps that were flush with the surrounding floor surface. Concrete drilling for the 2 test borings was accomplished by Omaha Concrete Sawing under contract to Lueder Construction using a 4-inch diameter core bit and a hammer drill. The 4-inch diameter drill-holes were protected after subgrade evaluations using temporary expanding plug.

Subgrade evaluation included observation of conditions below the floor slab, in-situ field testing at each drilled location, and laboratory testing on Shelby tube samples of the subgrade material at the 2 test boring locations. Observations were made by HDR and Thiele Geotech, Inc. (Thiele). Subgrade field and laboratory testing was performed solely by Thiele as directed by HDR.

Observation of the subgrade below the floor slab included direct visual observation through the open holes with the aid of a flashlight, close-up visual observations using a lighted, water-proof borescope lowered through the open drill-holes, and measurement of the floor slab thickness and depth to subgrade using a hooked probe made from #9 tie wire.

In-situ field tests on the subgrade consisted of static cone penetrometer (SCP) tests at each drilled location and a dynamic cone penetrometer (DCP) test performed at one drill-hole (1-12). Laboratory tests included moisture content and unit weight (wet and dry).

Thiele used a Humboldt Model H-4210A Portable Static Cone Penetrometer to perform the SCP tests. This device consists of a direct-read penetrometer that measures the amount of penetration resistance as the device is pushed into soil materials. The cone penetrometer was advanced into the subgrade by hand in continuous six-inch vertical intervals until refusal or the maximum vertical reach of the device was reached, whichever occurred first. Resistance readings were observed and recorded for each 6-inch interval. Measurements of the depth to subgrade at each location tested were also made using gradation markings on the cone penetrometer.

Thiele used a Humboldt Model H-4219 Heavy Duty Dual Mass Dynamic Cone Penetrometer to perform the DCP test in accordance with ASTM D6951/D6951M, "Standard Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications". Although not directly applicable to determining density of non-cohesive soils as would be the Standard Penetration Test (SPT), the DCP was used due to limited access and space of the Maintenance Shop hallway. The data obtained can be used to identify zones of relatively low density or consistency compared to the surrounding subgrade as stated in Note 1 of the ASTM Standard.

Laboratory test samples were collected by Thiele using thin-walled Shelby tubes at two test boring locations; one advanced about 2.3 feet west of Building Column MG-15 (identified as Boring No. ST-1) and one advanced about 2.3 feet east of Building Column MG-15 (identified as Boring No. ST-2). Borings were initiated using a 3-inch diameter hand auger to advance through about 6 inches of gravel comprising the upper-most portion of the floor slab subgrade. Below the gravel layer, continuous Shelby tube samples were collected to depths of about 4 feet below the top of subgrade where refusal was encountered on coarse gravel.

Our investigation also included review of a previous geotechnical investigation report prepared in support of design of the original Maintenance Shop structure (Geotechnical Services, Inc. 1977).

4.3.7.2 Results

Forensic investigation results including field observations, SCP and DCP test results, laboratory test results, and previous geotechnical investigation results are summarized below. Test reports by Thiele and the previous geotechnical investigation report are included in Attachment 6.

4.3.7.2.1 Observation Results

Visual observations and measurements were made as described above. Data obtained at each drill-hole are summarized below in Table 4-3.

Table 4-3 - Maintenance Shop Forensic Investigation Observations

Drill-hole Number	Floor Slab Thickness(1) (inches)	Depth to Subgrade from Top of Slab(1) (inches)	Void Space Depth(1) (inches)	Straight-Line Distance from Column MG-15(2) (feet)	Cardinal Direction Relative to Column MG-15 (based on plant north)	Comments
1-1	5.0	7.0	2.0	18.0	WSW	upper 6" granular fill(3)
1-2	5.3	6.0	0.8	18.0	WNW	upper 6" granular fill(3)
1-3	5.0	8.0	3.0	12.0	WSW	upper 6" granular fill(3)
1-4	5.0	8.5	3.5	11.5	W	fine-grained fill
1-5	5.5	7.5	2.0	9.5	NW	upper 6" granular fill(3)
1-6	5.0	9.5	4.5	7.5	SW	upper 6" granular fill(3)
1-7	5.5	8.0	2.5	5.5	W	upper 6" granular fill(3)
1-8	4.3	12.5	8.3	5.5	SSW	upper 6" granular fill(3)
1-9	5.0	11.5	6.5	2.0	NNW	upper 6" granular fill(3)
1-10	5.5	10.0	4.5	4.5	S	upper 6" granular fill(3)
1-11	5.0	13.5	8.5	3.5	N	upper 6" granular fill(3)
1-12	5.5	9.0	3.5	2.5	SE	upper 6" granular fill(3)
1-13	5.0	9.5	4.5	5.5	SE	upper 6" granular fill(3)
1-14	5.5	10.0	4.5	6.5	ESE	upper 6" granular fill(3)
1-15	6.0	8.5	2.5	11.5	ESE	upper 6" granular fill(3)
1-16	5.5	7.0	1.5	16.5	ESE	upper 6" granular fill(3)
EW-1	6.0	7.0	1.0	19.0	ENE	upper 6" granular fill(3)
EW-2	5.0	6.0	1.0	45.0	ENE	upper 6" granular fill(3)
EW-3	5.0	5.0	0.0	65.0	ENE	upper 6" granular fill(3)
EW-4	5.5	5.8	0.3	83.0	ENE	upper 6" granular fill(3)
NS-1	4.8	5.0	0.2	33.0	NNW	upper 6" granular fill(3)
NS-2	5.0	5.3	0.3	47.0	NNW	upper 6" granular fill(3)

Notes:

(1) Approximate value, rounded to the nearest 1/10-inch, based on probe measurements using a tape measure.

(2) Approximate value, rounded to the nearest ½-foot, based on scaled plan drawings; not field measured.

(3) Subjective apparent material encountered based on CPT probe action during advancement through subgrade is believed to consist of fine-grained fill.

N = north, S = south, E = east, W = west

As indicated by the data above, floor slab thickness at the locations drilled ranged from about 5 to 6 inches at the testing locations. Drilling completed to access the floor slab subgrade resulted in penetration slightly below the slab. Observations during drilling indicated that the drill usually advanced beyond the slab bottom about an inch. Measured void space of less than about an inch was not considered significant or representative of a void space.

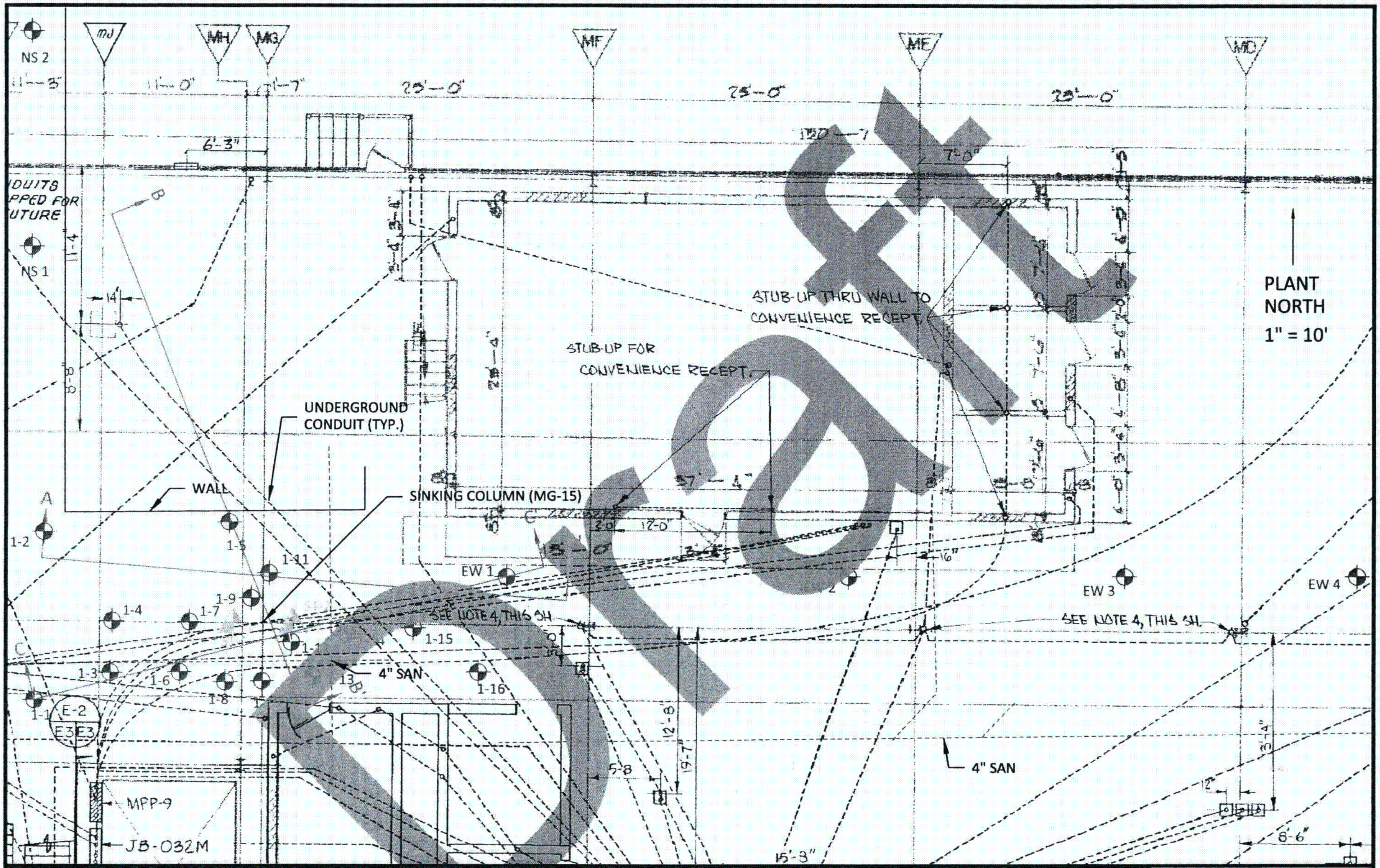
Significant void space (greater than about 1 inch) was detected below the floor slab at all but one of the 16 locations drilled in the open area surrounding Column MG-15. Away from the settled column in the adjacent hallways, no significant void space was detected below the floor slab in any of the locations drilled (drill-holes EW-1 through EW-4 and NS-1 and NS-2).

Overall, void depths ranged from zero to about 8.5 inches and averaged about 3.0 inches. In the area surrounding the settled column, void depths were greater ranging from about 0.8 to 8.5 inches and averaging about 3.9 inches.

The data also indicates that void space below the floor slab in the area investigated is greater nearer to the settled column as shown below.

- Within about 3.5 feet of the settled column, void space averaged 6.2 inches
- Within about 4.5 feet of the settled column, void space averaged 5.8 inches
- Within about 5.5 feet of the settled column, void space averaged 5.5 inches
- Within about 7.5 feet of the settled column, void space averaged 5.3 inches
- Beyond about 7.5 feet from the settled column, void space averaged 1.4 inches
- Beyond about 18 feet from the settled column, void space averaged 0.5 inches

The above statements related to void space are illustrated in cross sections A-A', B-B', and C-C' presented as Figures 4-12, 4-13, and 4-14. The Section locations are shown in Figure 4-11.



Note: Background is OPPD Drawing #16892

- Drilled Test Holes
- Drilled Test Holes with Shelby Tube Sampling
- Sections - See Figures 2-4

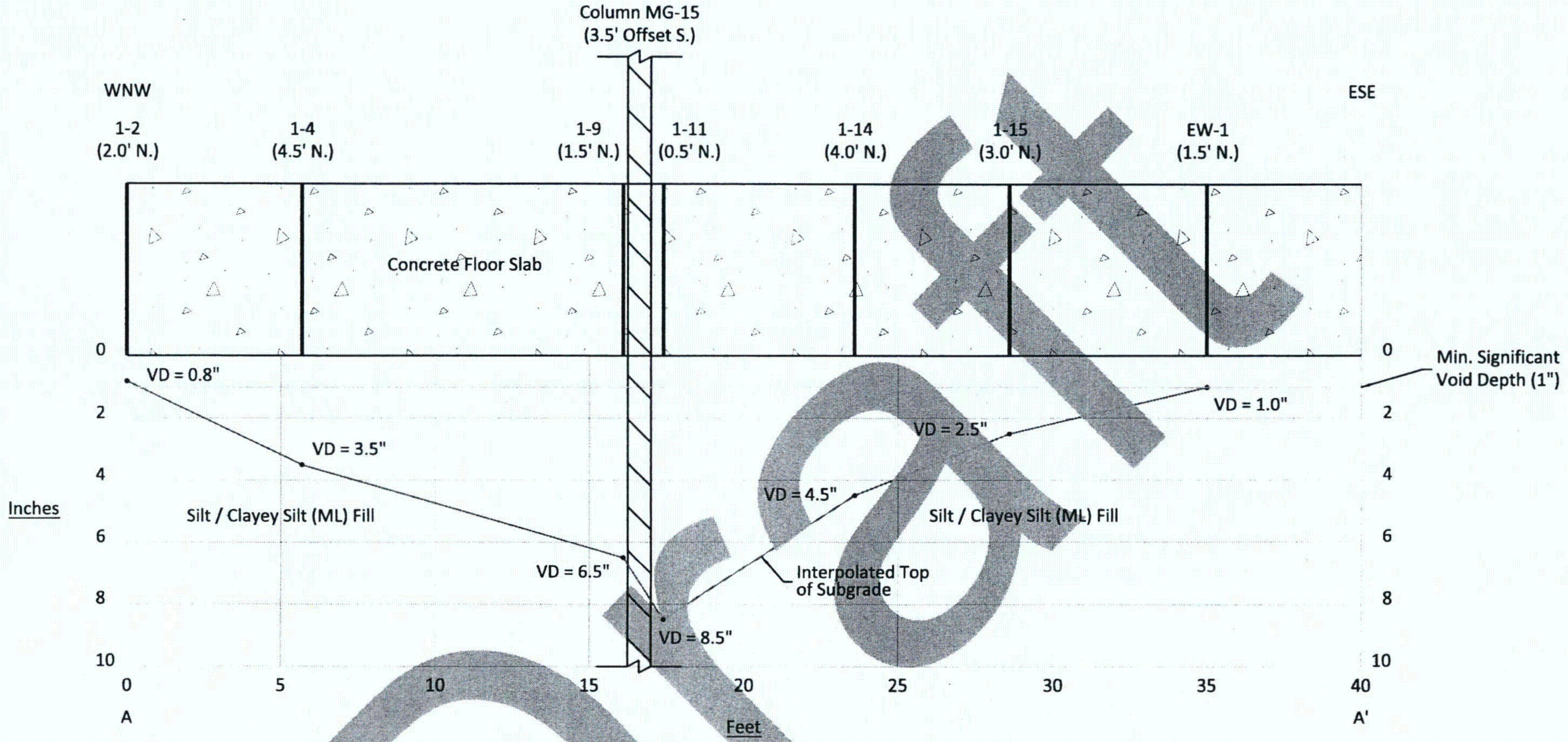
OPPD
Omaha Public Power District

Maintenance Shop Drilling Locations
Cross Section Location Plan
Sections A-A, B-B, C-C
Fort Calhoun Station

Plant and Facility Geotechnical and Structural Assessment

HDR

DATE	Dec 2011
FIGURE	4-11



LEGEND

•	Measured Point Depth
VD = 1.0"	Void Depth
1-2 (2.0' N.)	Drilled Hole Identification (Offset)

NOTES

Nominal floor slab thickness is 5.5".
Vertical Scale is 1" = 5".
Horizontal Scale is 1" = 5'.

Omaha Public Power District

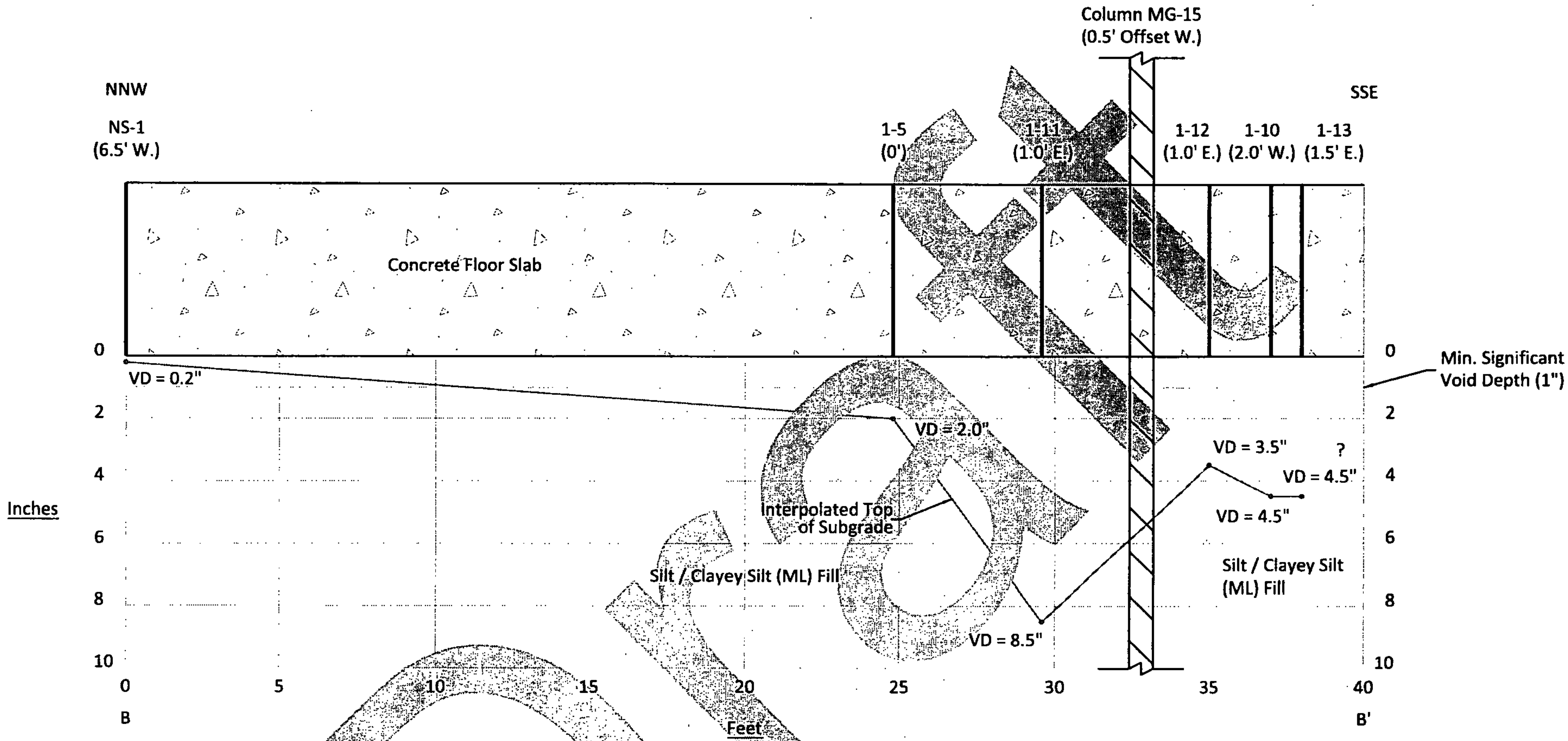
Maintenance Shop Drilling Locations
Cross Section A-A
Fort Calhoun Station

Plant and Facility Geotechnical and Structural Assessment

HDR

DATE
Dec 2011

FIGURE
4-12



LEGEND

•	Measured Point Depth
VD = 1.0"	Void Depth
1-2 (2.0' N.)	Drilled Hole Identification (Offset)
?	Extent of Void Unknown

NOTES

Nominal floor slab thickness is 5.5".
Vertical Scale is 1" = 5".
Horizontal Scale is 1" = 5'.

Omaha Public Power District

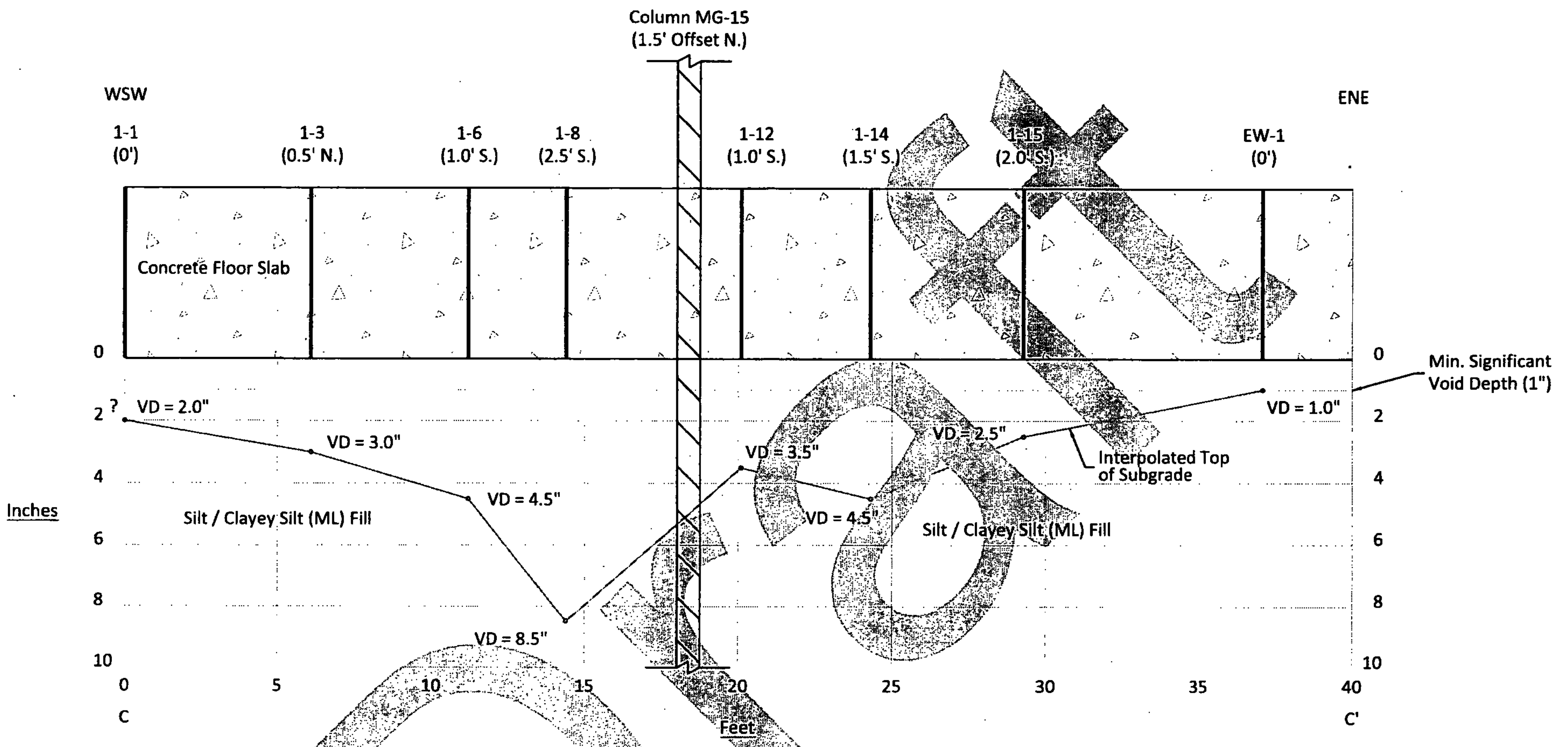
Maintenance Shop Drilling Locations
Cross Section B-B
Fort Calhoun Station

Plant and Facility Geotechnical
and Structural Assessment

HDR

DATE
Dec 2011

FIGURE
4-13



LEGEND

•	Measured Point Depth
VD = 1.0"	Void Depth
1-2 (2.0' N.)	Drilled Hole Identification (Offset)
?	Extent of Void Unknown

NOTES

Nominal floor slab thickness is 5.5".
Vertical Scale is 1" = 5".
Horizontal Scale is 1" = 5'.

Omaha Public Power District

Maintenance Shop Drilling Locations
Cross Section C-C
Fort Calhoun Station

Plant and Facility Geotechnical and Structural Assessment

HDR

DATE
Dec 2011

FIGURE
4-14

4.3.7.2.2 SCP Test Results

SCP tests were performed as described above. Data obtained at each drill-hole are summarized below in Table 4-4.

Table 4-4 - Maintenance Shop SCP Test Results

Drill-hole Number	Depth from Top of Subgrade (feet)	Gauge Reading (kg/cm^2)	Approx. Correlated N-value (uncorrected, blows/foot)	Relative Consistency or Firmness	Straight-Line Distance from Column MG-15[1] (feet)	Cardinal Direction Relative to Column TC-15 (based on plant north)	Comments
1-1	0.0-0.5	24.5	6	soft	18.0	WSW	refusal 9" BTOS
	0.5-1.0	40	10	stiff			
1-2	0.0-0.5	20	5	soft	18.0	WNW	refusal 7.5" BTOS
	0.5-1.0	51	13	stiff			
1-3	0.0-0.5	6	2	very soft	12	WSW	refusal 7 " BTOS
	0.5-1.0	53	13	stiff			
1-4	0.0-0.5	45	11	stiff	11.5	W	refusal 36" BTOS
	0.5-1.0	48	12	stiff			
	1.0-1.5	14	4	soft			
	1.5-2.0	36.5	9	stiff			
	2.0-2.5	31.5	8	med. stiff			
	2.5-3.0	51	13	stiff			
1-5	0.0-0.5	35	9	stiff	9.5	NW	refusal 7" BTOS
	0.5-1.0	55	14	stiff			
1-6	0.0-0.5	8	2	very soft	7.5	SW	refusal 8" BTOS
	0.5-1.0	53	13	stiff			
1-7	0.0-0.5	28	7	med. stiff	5.5	W	refusal 8" BTOS
	0.5-1.0	53	13	stiff			
1-8	0.0-0.5	54	14	stiff	5.5	SSW	refusal 4" BTOS
1-9	0.0-0.5	9	2	very soft	2	NNW	refusal 23" BTOS
	0.5-1.0	5	1	very soft			
	1.0-1.5	16	4	soft			
	1.5-2.0	30	8	med. stiff			
1-10	0.0-0.5	9	2	very soft	4.5	S	refusal 10" BTOS
	0.5-1.0	54	14	stiff			
1-11	0.0-0.5	52	13	stiff	3.5	N	refusal 1.5" BTOS
1-12	0.5-1.0	52	13	stiff	2.5	SE	refusal 6" BTOS
1-13	0.0-0.5	0	0	NA	5.5	SE	refusal 10" BTOS
	0.5-1.0	50	13	stiff			
1-14	0.0-0.5	0	0	NA	6.5	ESE	refusal 10" BTOS
	0.5-1.0	52	13	stiff			
1-15	0.0-0.5	30	8	med. stiff	11.5	ESE	refusal 6" BTOS

1-16	0.0-0.5	9	2	very soft	16.5	ESE	refusal 8.5" BTOS
	0.5-1.0	54	14	stiff			
EW-1	0.0-0.5	39	10	stiff	19.0	ENE	refusal 7" BTOS
	0.5-1.0	55	14	stiff			
EW-2	0.0-0.5	58	15	stiff	45.0	ENE	refusal 6" BTOS
EW-3	0.0-0.5	59	15	stiff	65.0	ENE	refusal 5" BTOS
EW-4	0.0-0.5	56	14	stiff	83.0	ENE	refusal 5" BTOS
NS-1	0.0-0.5	52	13	stiff	33.0	NNW	refusal 5" BTOS
NS-2	0.0-0.5	50	13	stiff	47.0	NNW	refusal 14.5" BTOS
	0.5-1.0	32	8	med. stiff			
	1.0-1.5	46	12	stiff			

Notes:
(1) Approximate values based on scaled investigation plan drawings; not field measured.
BTOS = below top of subgrade.
N = north, S = south, E = east, W = west

No voids were detected below the top of subgrade at the test locations surrounding the settled building column. Based on N-values (uncorrected) correlated from SCP test cone index results, medium stiff to stiff fine-grained fills were encountered at all test locations. Very soft to soft soils were sometimes encountered in the upper 6 inches of subgrade.

The Humboldt (manufacturer) user's manual for the SCP test device provides a coefficient of 0.25 for correlating the direct read value of cone index (Q_c) in kilograms per square centimeter to N-value. The Humboldt Manual states that the correlation was determined through extensive field use, but is not absolute, and should be verified for local soil types. Because of the hydro-excavation required in the upper 10 feet of all test borings completed during the geotechnical investigation, direct correlation with on-site soils was not possible. As such, the N-values provided in the data table above are not based on correlations with site soils, and were used only for qualitative comparison and evaluation in this investigation.

4.3.7.2.3 DCP Test Results

DCP testing was performed at drill-hole 1-12. The DCP test was performed from about 4.3 feet to 24.2 feet below the top of subgrade. No voids or very soft to soft soil zones were detected by the DCP test. Cone index correlated California Bearing Ratio (CBR) values ranged from about 6.5 to 50. Cone index correlated bearing capacity ranged from about 2,000 to 7,000 pounds per square foot. Anomalously high CBR and bearing capacity values were obtained for soil at about 60 inches below the top of subgrade; these values are related to unusually high blow counts believed to be the result of the cone tip encountering a particle of gravel and are not included in the CBR and bearing capacity ranges mentioned above.

4.3.7.2.4 Shelby Tube Samples and Laboratory Test Results

Soils encountered during Shelby tube sampling were generally logged as silt. At one 12-inch interval (1.0 to 2.0 feet below top of subgrade) the material encountered was logged as lean clay. Shelby tube advancement/recovery ranged from 8 to 18 inches. Refusal was encountered at both test borings at about 4 feet below top of subgrade on coarse gravel (crushed limestone) believed to be at the previous plant grade and placed during original plant construction to stabilize the ground surface for heavy equipment traffic.

Moisture contents of the sampled soils ranged from 17.2 to 24.8 percent and averaged 20.6 percent. Void ratio (based on an assumed specific gravity of 2.7) ranged from 0.591 to 0.731 and averaged 0.600. Percent saturation (based on an assumed specific gravity of 2.7) ranged from 78 to 96 and averaged 86.

Wet unit weight ranged from 118.8 to 125.9 pcf and averaged 123.6 pcf. Dry unit weight ranged from 97.3 to 105.9 pcf and averaged 102.5 pcf. Based on an assumed Standard Proctor dry density of 104 pcf, estimated relative compaction ranged from 94 to 100 percent and averaged 99 percent.

4.3.7.2.5 Previous Geotechnical Investigation Report

In 1977, a geotechnical investigation and report was completed by Geotechnical Services, Inc. (GSI) to support the foundation design of the Maintenance Shop building. The investigation included 4 test borings (one of which was advanced in the immediate vicinity of Building Column MG-15), field SPTs, collection of subsurface soils using thin-walled Shelby tube and standard split-spoon samplers, and laboratory testing. The investigation indicated that the southern portions of the planned building footprint (including soils upon which Building Column MG-15 is founded) consisted of 7 to 9 feet of loess derived fill that classified as medium stiff, low plasticity clayey silt (ML). Fine sand fill was encountered along northern portions of the planned building footprint. Below the fill, medium dense to dense stratified alluvium including sandy silty clays, fine sands, and thin clay seams were encountered. The report concluded the following:

- The building could be supported on shallow foundations.
- Existing site fills are suitable.
- Cohesive soils would provide a safe net (FS=3) allowable bearing pressure of 2,000 psf.
- Total settlement of ¾ to 1 inch would be expected.
- Settlement would be rapid and differential settlements would not be a problem.

4.3.7.3 Discussion and Conclusions

Observations of the conditions underlying the floor slab in the vicinity of Column MG-15 confirm that the subgrade has subsided and a void space has developed. The void space ranges from about none to 8.5 inches in depth below the bottom of the floor slab and extends about 15 feet to the north, east and west-northwest of Column MG-15. The lateral extent of void beneath the floor slab to the south, southeast and southwest was not determined by this investigation.

Field testing including SCP and DCP tests on the subgrade soil below the floor slab did not identify the presence of voids or soft soils below the top of subgrade at tested locations. Field observations and laboratory testing from this investigation and from the previous GSI investigation are in general agreement and indicate that the fine grained loess derived fill in the vicinity of Column MG-15 consists of medium stiff to stiff, low plasticity silt that has allowable bearing pressure of 2,000 psf or greater. Neither field observations nor field and laboratory testing performed for this investigation indicated poor fill placement or conditions that would result in subgrade or column settlement of the magnitude observed.

Based on these observations and field and laboratory test results, the subsidence and resultant void identified below the floor slab and the column settlement and apparently related settlement cracking expressed in nearby masonry walls is not attributed to the Triggering Mechanism of Soil Collapse (due to first time wetting). As such, the CPFMs associated with this Triggering Mechanism; 7a - Cracked Slab, Differential Settlement of Shallow Foundation, and Loss of Structural Support; 7b - Displaced Structure/Broken Connections, and 7c - General Site Settlement, are ruled out for the Maintenance Shop.

In addition to the distress in the Maintenance Shop, cracks have recently been observed and documented in the Technical Support Building in areas located south, southwest, and west of the Maintenance Shop distress area (area of subgrade void, settled column, and wall cracking). The results of the assessment for the Technical Support Center (see Section 5.5) indicate that this distress is associated with KDI #3. Therefore, the Triggering Mechanism of Soil Collapse and its associated CPFMs listed above are also ruled out for the Technical Support Center.

The results of this KDI #3 forensic investigation show that the Triggering Mechanism of Subsurface Erosion and Piping (due to pumping) is likely responsible for the subsidence and related void and settlement distress in the Maintenance Shop and the distress (cracked walls) in the Technical Support Center. Voids, material loss, and material movements have been identified by investigations in the Turbine Building Basement (KDI #1) including along the north wall of the Turbine Building which is a shared/adjoining wall with the Maintenance Shop. The voids/subgrade settlement and distress observed in the Maintenance Shop are believed to be directly related to deep subsurface piping erosion and soil losses occurring at and radiating out from damaged subfloor drain pipes in the Turbine Building subgrade.

Based on the KDI #1 investigation, it appears that material has been removed below the Turbine Building north wall through piping as a result of the hydraulic gradient created by the breaks in the subfloor drain pipes. Piping has been evidenced by depressed groundwater levels, measured voids below the Turbine Building basement floor slab, and sediment accumulated in the Turbine Building sump pit. The depressed groundwater levels and void conditions are more prominent near the northwest portion of the Turbine Building adjacent to the observed KDI #3 structural distresses. We presume that the piping and void conditions extend north beyond the extents of the Turbine Building basement floor slab and below portions of the Maintenance Shop (and Technical Support Center). The soil column above the presumed piping and void condition is thought to be subsiding as a block unit, or column and translating to the ground surface resulting in the void space observed below the floor slab. It should be noted that our investigation was not exhaustive. Subgrade void space was not delineated to the south, southeast, or southwest (see Figure 4-11), which are toward the locations of observed/measured groundwater flow, groundwater lows, and voids below the Turbine Building basement. It should also be noted that wall cracking expressed in the Maintenance Shop masonry walls of the Men's restroom appears to be expanding (crack aperture appears larger than previously noted during structure observations in August/September 2011).

4.3.7.4 Recommendations

HDR's recommendations are listed below.

- The results of the KDI #3 forensic investigations have found that the distress observed in both the Maintenance Shop (failed column) and the Technical Support (cracked walls) are not associated with the Triggering Mechanism 7 - Soil Collapse (due to first time wetting). Therefore the CPFMs associated with this Triggering Mechanism (7a-7c) have been ruled out by this forensic investigation. The results show that the distress in both the Maintenance Shop and the Technical Support Center are connected to KDI #1, which is associated with the uncontrolled drainage of the groundwater into the broken Turbine Building basement drainage system piping. KDI #1 is associated with the Triggering Mechanism of Subsurface Erosion/Piping (due to pumping) and the CPFM applicable to the Maintenance Shop and Technical Support Center is 3a - Undermining and settlement of shallow foundation/slab (due to pumping). This CPFM will only be ruled out when the physical modifications presented for KDI #1, as presented in Section 4.1 of this Assessment Report, are implemented.
- It is recommended that OPPD implement physical modifications to remediate the distress in the Maintenance Shop as planned (helical piers and jacking). This may or may not affect adjacent masonry walls exhibiting settlement cracking. However, this does nothing to mitigate the likely cause of the observed Maintenance Shop distresses, Subsurface Erosion and Piping. Nor does it ensure that further distress will not be realized in other structural components of the building in nearby areas that may also be affected by Subsurface Erosion and Piping but not yet expressing any observable distress. Future distress could include other building support columns, the adjacent elevator shaft, and other nearby masonry walls (load bearing or not). It should be noted that our investigation did not determine the extent of settlement or voids to the south, southeast, or southwest of settled Column MG-15.
- No further investigations are recommended for the purposes of this Assessment Report. However, further investigations could be undertaken by the owner as part of the design for the remedial work to repair the Maintenance Shop and Technical Support Center distress. This could include investigation of the subgrade below the floor slab in the Maintenance Shop to the south, southeast, and southwest of Column MG-15 to include drilling, coring, SCP and DCP tests, soil sampling, and laboratory testing as appropriate to delineate the area of subsiding subgrade and identify other structural building elements at risk. It is further recommended that the physical modifications outlined in the KDI #1 forensic investigations be competed before the physical modifications to remediate the distress in the Maintenance Shop and Technical Support Center are implemented. This is to ensure that the subsurface erosion/piping associated with the broken pipes under the Turbine Building basement slab is halted. Continued subsurface erosion/piping would most likely reduce the efficacy of any physical modifications designed to remediate the distress in the Maintenance Shop and the Technical Support Center.

4.4 Comparative Evaluation of Geotechnical Analyses

The purpose of this comparative evaluation is to assess the potential impacts of the 2011 flood on the overall geotechnical conditions at the FCS site. This assessment included a comparative evaluation of new and existing geotechnical data in an attempt to assess whether the foundation soils have been disturbed or weakened from the sustained high water.

The primary basis of comparison for this evaluation was provided by 1) the penetration resistance data recorded during drive sampling and seismic refraction surveys completed as a part of the pre-flood investigations, and 2) the subsurface investigations conducted for this assessment. The penetration resistance data from these investigations provide an indirect but useful indication of the relative strength and stiffness of the subsurface soils and bedrock at the FCS site. The seismic refraction surveys provide an estimation of the p-wave (compression) wave velocity, which can be an additional indication of the relative strength and stiffness of these materials.

4.4.1 Site Conditions

Site grades before construction at the FCS site generally ranged from about el. 995 to 1000 ft. At the boring locations for the current investigations, the site grades varied from el. 1000 to 1005 ft.

For reference, the generalized subsurface profile at the FCS site consists of the following strata in descending order:

- A 1- to 10-foot-thick layer of existing earth fill, most of which was placed at the time of the original construction
- An intermittent layer of soft to firm, fine alluvium (silts and clays) that varies in thickness from 0 to 20 feet
- A 50- to 60-foot-thick layer of loose to dense, coarse granular alluvium (primarily silty to poorly graded sands with some clay seams)
- Limestone/shale bedrock at depth of about 75 feet below present grades, or at about el. 930 ft.

The granular alluvium at the FCS site is comprised of a loose to medium dense layer of recent alluvium extending to about el. 960 ft and underlain by a dense older alluvium extending to the top of rock.

Groundwater levels at the times of the pre-flood and current investigations varied in elevation from about 986 to 1001 ft. River levels during the 2011 flood reached a high water elevation of approximately 1006.9 ft.

Additional discussions of the geological and geotechnical conditions at the FCS site are provided by Dames & Moore (1967, 1968) and HDR (2011).

4.4.2 Pre-Flood Investigations

4.1.1.1 Dames & Moore Drive Sampling

The majority of the geotechnical data obtained from the pre-flood investigations was derived from the subsurface investigation completed by Dames & Moore (D&M, 1967, 1968) of New York, NY. This investigation consisted of advancing 73 test borings in the area of the main facility using 3.25-inch diameter hollow stem augers and drive sampling at 5-foot intervals in the overburden soils and BX-size rock coring in the bedrock. The depths of the borings ranged from 50 to 150 feet below grade.

The Dames & Moore "Type U" sampler and the Standard Penetration Test (SPT) sampler were used to collect samples and measure driving resistance as the number of hammer blows per foot of sampler penetration. The D&M sampler retrieved 2.42-inch diameter drive samples using a 300 to 350-pound weight falling a vertical distance of 24 inches (energy = 600 to 680 foot-pounds). The SPT sampler retrieved 1.375-inch diameter drive samples using a 140-pound weight falling from a vertical distance of 30 inches (energy = 350 foot-pounds).

4.4.2.1 Dames & Moore Seismic Refraction Surveys

Three deep and one shallow seismic refraction surveys were also conducted as a part of the 1967 D&M investigation. The deep seismic surveys, which ranged in length from 600 to 1000 feet, were conducted to investigate the depth to rock at the site, as well as to estimate the p-wave velocities of the overburden soil and underlying bedrock. The shallow survey was 60 feet in length and was conducted to investigate an anomaly identified in the deep surveys that was determined to be a group of timber piles.

The results of the seismic refraction surveys from the D&M investigation are presented in Attachment 5A. A summary of the estimated p-wave velocities are presented in Attachment 5A, D&M Plate II C-1. The estimated p-wave velocities were found to vary from 1000 to 2000 feet/second between depths of 0 to 46 feet (or from about elevation 950 to 960 feet), and 6000 feet/second between the depths of 46 to 70 feet (from about elevation 930 feet). The p-wave velocity in the bedrock was estimated to be 15,300 feet/second.

4.4.2.2 Other Pre-Flood Investigations

The results of other pre-flood investigations conducted at the FCS were also used in this evaluation, including the following studies:

- The 1987 investigation by Woodward-Clyde Consultants (WCC, 1987), consisting of 21 borings at the site of the Training Center.
- The 1989 investigation by Woodward-Clyde Consultants (WCC, 1989), consisting of 14 borings at the site of the Administration Building.
- The 2003 investigation completed by Shaw Stone & Webster (SS&W, 2003), consisting 9 test borings and eleven (11) CPTs at the site of ISFSI.

4.4.2.3 Boring Location Plans and Logs

The logs of the borings from the pre-flood investigations are not included as a part of this technical memorandum. However, a tabular summary of the pertinent penetration data from the pre-flood investigations are provided in Attachment 5B. A Boring Location Plan for all of the pre-flood investigations is provided in Attachment 5B, Figure 1.

4.4.3 Current Investigation

4.1.1.1 General

The current subsurface investigation was completed by Thiele Geotech of Omaha, NE, in September of 2011. This investigation consisted of advancing 9 test borings and 12 cone penetration tests (CPTs) across the entire site. The locations of the borings and CPTs for the current investigation are shown in Attachment 5B, Figure 1.

Prior to commencing each boring and CPT, OPPD required that the upper 10 feet of soil be hydro-excavated (soil removed by jetting and vacuuming) due to the potential for encountering shallow utilities across the site. As a result, no soil samples or geotechnical data were retrieved from the present ground surface to a depth of 10 feet at these locations.

Detailed discussions of the current investigation are provided in HDR (2011).

4.4.3.1 SPT Sampling

The test borings were advanced to depths of 46 to 76 feet (refusal on rock) below grade. Sampling of the overburden soils was completed using 3.25-inch diameter hollow stem augers and drive sampling using the SPT sampler (ASTM D 1586-08a). Stability of the borehole was maintained during drilling with the use of bentonite slurry.

Prior to drilling and sampling, the SPT hammer systems on all drill rigs to be used in this investigation were energy calibrated by Foundation Testing and Consulting of Overland Park, KS. The calibration was performed to determine the actual efficiency in transferring the energy of the hammer blow to the drive sampler. The results of this calibration indicated that the efficiency of the SPT hammers used in the current investigation ranged from 77% to 83% (FTC, 2011).

4.4.3.2 Cone Penetration Testing

The CPTs completed by Thiele Geotech were advanced using an acoustic piezocone rig manufactured by Geoprobe Systems. The procedures for the CPT were performed in accordance with ASTM D 5778-07. The CPTs were advanced to depths of 16 to 47 feet below existing grade. Some of the CPTs reached refusal to further penetration in the dense alluvium at about elevation 960 feet.

The CPT field data for tip resistance (q_c), sleeve friction (f_s) and pore pressure (U) are provided in CPT-Pro software format in Attachment 5B.

4.4.3.3 Geophysical Investigations

The current investigation also included several geophysical testing methods conducted by Geotechnology, Inc. of St. Louis, MO. These methods included five seismic refraction lines and a series of ground penetrating radar and spontaneous resistivity grids. The purpose of the geophysical testing, including the seismic refraction surveys, was to investigate the overburden soils in an attempt to identify the presence of soft or loose zones of soil or voids that may have developed at the site from the flooding.

Locations of the seismic refraction surveys are provided in Attachment 5B, Figure 2. A full version of the report is provided in Geotechnology (2011).

The graphical results of the seismic refraction surveys from Geotechnology are provided in Attachment 5C and consist of plots of p-wave (compression) wave velocity versus depth along each of the seismic lines. The plots display contours of the p-wave velocities that range from about 1000 feet/second near the surface to about 6000 feet/second at the base of the alluvium. In general, the magnitude of the p-wave velocities were found to increase with depth, except some isolated zones of lower velocity material were encountered at depths of 40 to 75 feet below existing grade.

4.4.4 Interpretation of Penetration Resistance Data

Because the drive sampling in the pre-flood and current investigations were completed under variable site conditions and with different equipment and technology, the penetration resistance values (in number of sampler blows per foot of penetration) had to be adjusted for these differences to allow a reasonable basis for comparison. Correction factors were applied to the field blowcount values to account for these differing conditions in accordance with ASTM D 6066-96. These factors included corrections for:

- *Overburden pressure at the time of drive sampling*, since the grade of the site was raised about 10 feet since the 1967 D&M investigation, and that groundwater elevation and resulting effective stress condition varied at the time of each investigation;
- *Hammer energy*, since drive sampling with the D&M sampler applied a greater amount of energy than the SPT procedure (600-680 foot-pounds versus 350 foot-pounds);
- *Borehole diameter*; since sampling in larger diameter holes is slightly less efficient in transferring the energy to the sampler than in smaller holes;
- *Drill rod length*; since drive samples taken with shorter lengths of drill rod are more efficient than sampling with longer sections; and
- Whether *thin-walled liners* were utilized in the sampling process, since the use of liners is less efficient than sampling without liners.

As described above, each of these variables affect the magnitude of the field measured blowcount value recorded at the time of the investigation. Following the application of these correction factors, a normalized resistance value is developed for a hammer efficiency of 60%. This normalized value is referred to as $N1_{60}$. An efficiency of 60% was selected for the normalized value since the commonly used safety hammer with a rope-cathead has an efficiency of about 60%. In addition, many of the published correlations of SPT values and soil properties have been developed using blowcount data from this hammer system. Newer equipment that utilizes an automatic trip hammer typically has a higher efficiency (70 to 80%) than the rope-cathead system. Donut-type hammers typically have a lower efficiency that range from about 40 to 50%.

Calculations of the $N1_{60}$ values and background information for the correction factors are provided in Attachment 5B. Plots of the $N1_{60}$ values versus elevation of the recorded blowcount for the pre-flood and current investigations are presented in Attachment 5B, Figure A-3. Using the correlations and procedures recommended by

Robertson et al (1986) and Lunne et al (1997), estimates of $N1_{60}$ were derived from the CPT data and these data points have been included in these plots.

As depicted in Attachment 5B, Figure A-3, the $N1_{60}$ values from the pre-flood and current investigations show a similar pattern and scatter of blowcounts that range from 2 to 60 blows/foot along the full depth of the subsurface profile. The mean and standard deviation for the pre-flood and current $N1_{60}$ values are plotted in Attachment 5B, Figures A-3 and A-4 along the full depth of the profile. These plots also display a similar range and scatter of values.

4.4.5 Preliminary Findings and Conclusions

Comparison of the computed $N1_{60}$ values for the pre-flood and current investigations indicate that there was no observable difference in the overall geotechnical conditions at the site and that the foundation materials have not been disturbed or significantly weakened from the flood inundation. In addition, comparison of the seismic refraction data from the pre-food and current investigations reveals similar magnitudes of p-wave velocities over the full depth of the overburden soils, and no observable differences were identified from this work. The presence of loose to medium dense zones with lower p-wave velocities interbedded within denser materials confirm the inherent variation in the resistance data retrieved in the pre-flood and current investigations.

Based on these findings and evaluations, it appears that the overall geotechnical conditions at the site have not been significantly altered due to the sustained high water. The observed scatter of data points in both plots is consistent with the relatively wide range of strength and stiffness and corresponding blowcounts typically encountered in the alluvial soils within the Missouri River valley.

It should be noted that the findings and conclusions from the SPT comparative analysis are considered applicable only to those existing soils below a depth of 10 feet at the site, or below an elevation of about 995 feet, since these soils were hydro-excavated to avoid damaging buried utilities. The upper 10 feet of soil may have been disturbed from underseepage and high exit gradients beneath the temporary levees during high water. Additionally, disturbance to the site could have been resulted from the settlement of utility backfill during drawdown of the river level and groundwater.

It should also be noted that additional test borings are planned at the site and the data from this SPT sampling will be incorporated into this study when available.

Additionally, it should be noted that these findings and conclusions are not applicable to the potential impacts that may have occurred due to the presence of the broken piping and groundwater flow into the Turbine Building Sump since the time of our site visits and current investigations.

4.4.6 Limitations

This Assessment Report presents the preliminary findings and conclusions for an engineering evaluation of the potential impacts of the 2011 Flood on the geotechnical conditions at the FCS site. It has been prepared in accordance with generally accepted engineering practice and in a manner consistent with the level of care and skill for this type of project within this geographical area. No warranty, expressed or implied, is made.

Geotechnical engineering and the geologic sciences are characterized by uncertainty. The professional judgments presented herein are based on our review of available design and construction information provided to us, the results of field exploration and laboratory materials testing by others, the results of engineering evaluations, our general experience and the state-of-the-practice at the time of this writing.

4.4.7 Test Standards

- ASTM D 1586-08a, "Standard Test Method for Penetration Test (SPT) and Split-Barrel Sampling of Soils."
- ASTM D 5778-07, "Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils."
- ASTM D 6066-96, "Standard Practice for Determining the Normalized Penetration Resistance of Sands for Evaluation of Liquefaction Potential."

SECTION 5.0
PRIORITY 1 STRUCTURES

Section 5.1
Intake Structure

5.1 Intake Structure

5.1.1 Summary of Intake Structure

Baseline information for the Intake Structure is provided in Section 2.0, Site History, Description, and Baseline Condition.

The Intake Structure is located at the extreme east side of the PA and is constructed directly into the riverbank. This structure is subjected to unbalanced soil loads from east to west with the basement walls backfilled on the west and subjected to the river on the east face.

The Intake Structure is a multi-floored Class I structure below operating floor el. 1007.5 ft. From foundation mat at approximately el. 960.8 ft to el. 1014.5 ft, the structure is cast-in-place reinforced concrete with integral pilasters that align with the steel columns above grade. The basement walls are braced by vertical concrete walls and horizontal floor slabs. The mat foundation is on 20-in.-diameter steel pipe piles driven to bedrock. From el. 1014.5 ft to the roof at approximately el. 1035.6 ft, the structure is a braced rigid steel frame clad with Ar-lite precast concrete sandwich panels. The roof is a multi-layer built-up roof supported by metal decking spanning between open-web steel joists. The roof structure is seismically braced independent of the metal deck.

The Intake Structure houses major systems and components, both CQE and non-CQE, in designated rooms. The major function of the Intake Structure is to provide water from the Missouri River that is required for component cooling and fire fighting at Fort Calhoun Station, and to provide the structural support and environmental protection necessary to ensure the functional integrity of the CQE systems and components under operational and environmental conditions.

5.1.2 Inputs/References Supporting the Analysis

Table 5.1-1 lists references provided by OPPD and other documents used to support HDR's analysis.

Table 5.1-1 - References for Intake Structure

Document Title	OPPD Document Number (if applicable)	Date	Page Number(s)
System Design Basis Document	SDBD-STRUC-503 Rev. 10	6/22/2010	46, 57–61
2009 Structural Inspection of the Intake Building and Misc. Areas	SE-PM-AE-1002	7/16/2009	All
Incident Report Summary	CR 2011-5369	6/5/2011	All
Incident Report Summary	CR 2011-5254	6/1/2011	All
Incident Report Summary	CR 2011-5321	6/3/2011	All
Incident Report Summary	CR 2011-5323	6/3/2011	All
Incident Report Summary	CR 2011-5377	6/5/2011	All
Incident Report Summary	CR 2011-5384	6/6/2011	All
Incident Report Summary	CR 2011-5473	6/10/2011	All
Incident Report Summary	CR 2011-5737	6/22/2011	All
Incident Report Summary	CR 2011-5805	6/26/2011	All
Incident Report Summary	CR 2011-5932	7/1/2011	All

Table 5.1-1 - References for Intake Structure			
Document Title	**OPPD Document Number (if applicable)**	**Date**	**Page Number(s)**
Summary of Vibroflotation		1/27/1972	All
Naval Facilities Engineering Command, Design Manual 7.01, Soil Mechanics		9/1986	All

Detailed site observations—field reports, field notes, and inspection checklists—for the Intake Structure are provided in Attachment 8.

Observed performance and pertinent background data are as follows:

- The foundation slab is cast with the pipe piles embedded within the concrete mass to provide a "fixed head" condition for the pile design (see SDBD-STRUC-503).
- The piles consist of 20-in.-outside-diameter pipe with 1.031-in. wall thickness, which meets American Petroleum Institute (API) standard 5L Grade B (minimum yield stress [F_y] = 35 kips per square inch [ksi]). The piles were driven open-ended into sound bedrock and load tested for compression, tension, and lateral loads. The pile design allowed for .0625-in. corrosion of the wall thickness and was considered to be an additional level of conservatism due to the cathodic protection system (see USAR-5.7).
- The coefficient of lateral subgrade modulus is based on lateral load testing of test piles in the in situ soil condition, which is conservative because the granular soils were subsequently compacted.
- The in situ granular soils were compacted via vibroflotation to minimize the possibility of seismic liquefaction (see Summary of Vibroflotation).
- For the majority of the flood event, the structure was protected by a sandbag wall along the entire west face combined with interior sandbags and portable pumps located at the exterior doors on the east and west walls.
- The riverbank surrounding the structure is protected by revetment and slopes downward at 3H:1V.
- The Trenwa cable trench connects to the north and south sides of the structure.
- Concrete-encased service lines, including the Raw Water return line, connect to the south side of the structure.
- The Raw Water supply line and two Fire Protection lines connect to the north side of the building.
- Incident report summaries listed in Table 5.1-1 document many areas of the structure where groundwater has infiltrated the building through previously monitored cracks in the concrete, through wall penetrations, and through conduit.
- The Intake Structure is designed to withstand an external hydrostatic load due to flooding of the Missouri River to el. 1014 ft (see SDBD-STRUC-503).
- Without special provisions, the Intake Structure can accommodate flood levels of up to 1004.5 ft without water entering the structure. For higher flood levels, protection can be provided by steel flood barriers equipped with seals (up to el. 1009.5 ft) and sandbags and other methods to el. 1014 ft (see SDBD-STRUC-503).
- The building was located outside the Aqua Dam perimeter and was protected by the steel flood barriers and sandbags, with small portable pumps to remove light water infiltration.
- A layer of dried river sediment was present on the north and south grades adjacent to the structure.
- Small localized areas directly at the soil and exterior wall interface had signs of subsidence and scour. However, globally there were no signs of large-scale soil movement.
- Visual observation was not made to the river (east) side of the structure due to high water levels in the Missouri River at the time of the field inspection.

- General observations of the interior of the structure indicated minor concrete cracking with both current water infiltration (damp to slight running water) and dry walls with signs of water infiltration that occurred at an earlier time. The observed cracking appears to be a condition previously recorded and monitored.

5.1.3 Assessment Methods and Procedures

5.1.3.1 Assessment Procedures Accomplished

Assessments of the Intake Structure included the following:

- Visual inspection of the interior of the structure with the exception of the recirculation tunnel, the north stairwell, and the operating floor at el 1993.5 ft.
- Visual inspection of the exterior of the structure where accessible. Inspection of the river (east) side of the structure was not possible due to high river levels at the time of the inspection.
- An assessment of collected survey data to-date for indications of trends in the movement of the structure.
- A review of previously referenced documents listed in Table 5.1-1.

Additional investigations were performed. These included the following non-invasive geophysical and invasive geotechnical investigations:

- Seismic surveys (seismic refraction and refraction micro-tremor) in the protected area. (Test reports were not available at the time of Revision 0.)
- Geotechnical test borings in the protected area. Note that OPPD required vacuum excavation for the first 10 ft of proposed test holes to avoid utility conflicts. Therefore, test reports will not show soil conditions in the upper 10 ft of test boring logs. (Test reports were not available at the time of Revision 0.)
- Inclinometer readings along the river that will provide an indication of slope movement were not installed at the time of Revision 0.

5.1.3.2 Assessment Procedures Not Completed

No additional assessment procedures have been identified for this structure.

5.1.4 Analysis

Identified PFMs were initially reviewed as discussed in Section 3.0. The review considered the preliminary information available from OPPD data files and from initial walk-down observations. Eleven PFMs associated with five different Triggering Mechanisms were determined to be "non-credible" for all Priority 1 Structures, as discussed in Section 3.6. The remaining PFMs were carried forward as "credible." After the design review for each structure, the structure observations, and the results of available geotechnical, geophysical, and survey data were analyzed, a number of CPFMs were ruled out as discussed in Section 5.1.4.1. The CPFMs carried forward for detailed assessment are discussed in Section 5.1.4.2.

5.1.4.1 Potential Failure Modes Ruled Out Prior to the Completion of the Detailed Assessment

The ruled-out CPFMs reside in the Not Significant/High Confidence category and for clarity will not be shown in the Potential for Failure/Confidence matrix.

Triggering Mechanism 2 – Surface Erosion
CPFM 2b – Loss of lateral support for pile foundation

Reasons for ruling out:

- The pile foundation is located below basement elevation of approximately 974.7 ft, while grade is at approximate el. 1004.0 ft. Field observations of surface erosion have been isolated to a fencepost at the river's edge.
- The bathymetric survey did not indicate significant surface erosion at the east side, which was under water.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3b – Loss of lateral support for pile foundation (due to pumping)

Reasons for ruling out:

- There is no condition at this structure where pumping of water would result in differential head, thus resulting in loss of lateral support for the pile foundation.
- The small portable pumps that were used in this area removed surficial seepage, which would not create subsurface erosion or piping.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3c – Loss of lateral support for pile foundation (due to river drawdown)

Reason for ruling out:

- The pile foundation is located below basement elevation of approximately 974.7 ft, which is well below designated normal or low river levels. Therefore, the pile foundation is below the river level regardless of the rate of drawdown. Soil material around the piles will not be drawn upward as the river level subsides.

Triggering Mechanism 4 – Hydrostatic Lateral Loading (water loading on structures)
CPFM 4c – Wall failure in flexure
CPFM 4d – Wall failure in shear
CPFM 4e – Excess deflection

Reasons for ruling out:

- The Intake Structure is designed to withstand an external water load due to flooding of the Missouri River to el. 1014 ft (see SDBD-STRUC-503). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.

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- The structure cannot slide or overturn due to hydrostatic lateral loads because these loads are approximately equal on all sides of the structure.
- Visual observations did not identify distress to the structure that can be attributed to this PFM.

Triggering Mechanism 5 – Hydrodynamic Loading

CPFM 5a – Overturning
CPFM 5b – Sliding
CPFM 5c – Wall failure in flexure
CPFM 5d – Wall failure in shear
CPFM 5e – Damage by debris
CPFM 5f – Excess deflection

Reasons for ruling out:

- The Intake Structure is designed to withstand an external water load due to flooding of the Missouri River to el. 1014 ft (see SDBD-STRUC-503). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.
- The reinforced concrete walls of the Intake Structure were originally designed to withstand blast forces, making the likelihood of damage from floating debris small.
- Visual observations did not identify distress to the structure that can be attributed to these CPFMs.

Triggering Mechanism 6 – Buoyancy, Uplift Forces on Structures

CPFM 6a – Fail tension piles
CPFM 6b – Cracked slab, loss of structural support
CPFM 6c – Displaced structure/broken connections

Reasons for ruling out:

- The Intake Structure is designed to withstand an external hydrostatic load due to flooding of the Missouri River to el. 1014 ft (see SDBD-STRUC-503). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.
- Visual observations and survey measurements indicate no structure movement. Therefore, failed tension piles (CPFM 6a) and displaced structure and damage (CPFM 6c) did not occur.

Triggering Mechanism 7 – Soil Collapse (first time wetting)

CPFM 7b – Displaced structure/broken connections
CPFM 7c – General site settlement
CPFM 7d – Piles buckling from down drag

Reason for ruling out:

- The Intake Structure is directly adjacent to the Missouri River. The soil surrounding the structure, including the subgrade under buried utilities leading to the structure, is normally in a saturated condition.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10b – Displaced structure/broken connections

Reasons for ruling out:

- Permanent equipment that has the capacity to produce significant dynamic forces due to vibration is mounted on the base mat foundation slab of the structure. This structure is below the river level regardless of the flood elevation.
- Temporary pumping equipment located on the ground within the Aqua Dam perimeter produced minimal localized vibrations, which were offset from the structure and therefore deemed to have inconsequential effect.
- No broken structural connections or structural displacement was observed.
- This is not a changed condition due to the flood. The Intake Structure has been in service 38 years under similar saturated soils and machine vibration.
- The in situ granular soils were compacted via vibroflotation to minimize the possibility of liquefaction.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10c – Additional lateral force on below-grade walls

Reasons for ruling out:

- Permanent equipment that has the capacity to produce significant dynamic forces due to vibration is mounted on the base mat foundation slab of the structure. This structure is below the river level regardless of the flood elevation.
- Temporary pumping equipment located on the ground within the Aqua Dam perimeter produced minimal localized vibrations, which were offset from the structure and therefore deemed to have inconsequential effect.
- This is not a changed condition due to the flood. The Intake Structure has been in service 38 years under similar saturated soils and machine vibration.
- The in situ granular soils were compacted via vibroflotation to minimize the possibility of liquefaction.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10d – Pile/pile group instability

Reasons for ruling out:

- Permanent equipment that has the capacity to produce significant dynamic forces due to vibration is mounted on the base mat foundation slab of the structure. This structure is below the river level regardless of the flood elevation.
- Temporary pumping equipment located on the ground within the Aqua Dam perimeter produced minimal localized vibrations, which were offset from the structure and therefore deemed to have inconsequential effect.
- This is not a changed condition due to the flood. The Intake Structure has been in service 38 years under similar saturated soils and machine vibration. Reviewed condition survey reports do not indicate signs of distress that would be attributed to pile instability.
- The in situ granular soils were compacted via vibroflotation to minimize the possibility of liquefaction.

Triggering Mechanism 11 – Loss of Soil Strength due to Static Liquefaction or Upward Seepage

CPFM 11b – Displaced structure/broken connections
CPFM 11c – Additional lateral force on below-grade walls
CPFM 11d – Pile/pile group instability

Reasons for ruling out:

- The Intake Structure is located outside the Aqua Dam perimeter and was subjected to floodwater and is therefore not subjected to static liquefaction.
- This is not a changed condition due to the flood. The Intake Structure has been in service 38 years under similar saturated soils.

Triggering Mechanism 13 – Submergence

CPFM 13b – Corrosion of structural elements

Reasons for ruling out:

- The Intake Structure is directly adjacent to the Missouri River. The soil surrounding the structure, including the subgrade under buried utilities leading to the structure, is normally in a saturated condition.
- This is not a changed condition due to the flood. The Intake Structure has been in service 38 years under similar saturated soils. Reviewed condition survey reports do not indicate signs of distress that would be attributed to corrosion due to submergence.

Triggering Mechanism 14 – Frost Effects

CPFM 14a – Heaving, crushing, or displacement

Reason for ruling out:

- The Intake Structure is not susceptible to frost, and susceptible connecting utilities are below frost level.

5.1.4.2 Detailed Assessment of Credible Potential Failure Modes

The following CPFMs are the only CPFMs carried forward for detailed assessment for the Intake Structure as a result of the 2011 flood. This detailed assessment is provided below.

Triggering Mechanism 12 – Rapid Drawdown

CPFM 12a – River bank slope failure and undermining surrounding structures
CPFM 12b – Lateral spreading

The Triggering Mechanism and CPFMs could occur as follows: the river level drops faster than pore water pressure in the soil can dissipate. The saturated soil is elevated above the dropping river level. The sloped bank of the river provides no lateral pressure support for the saturated soil. At some point, there is insufficient support on the river side to support the saturated soils. At that point, the soils experience slope movements or even failure. Generally, slope failures associated with rapid drawdown are relatively localized and shallow in nature; however, deeper failures can occur.

Floodwater elevations, at the time of HDR's inspection, were above finished floor elevations, and river levels were being lowered at a relatively slow pace. River elevations were still well above normal levels. The drop in elevation of the river is expected to occur at a higher rate than the drop in elevation of the groundwater. This will result in an increased groundwater gradient. This increase could cause localized riverbank slope failure and/or lateral spreading.

At the time of Revision 0, the river level had dropped to a nominal normal level (roughly el. 994 ft). Field observation of the river bank area has not been performed since the river level dropped.

Adverse (Degradation/Direct Floodwater Impact More Likely)	Favorable (Degradation/Direct Floodwater Impact Less Likely)
The Intake Structure is in close proximity to the river.	Drawdown conditions required to trigger this CPFM had not occurred at the time of the field report. Therefore, field observations and data that discredit this CPFM could not be made.
Utilities provide many potential flow paths to and around the structure.	Soils in the area of the Main Underground Cable Bank and to the east are backfill materials that were placed and compacted during construction of site improvements and therefore would be expected to be less susceptible to rapid drawdown impacts.
Elevated saturated soils and elevated floodwater levels provide a water source. A potential path for water and soil migration can extend under the structure to the river, causing adverse effects attributed to river drawdown.	The river bank south of the Intake Structure is protected with riprap.
	The river bank to the north of the Intake Structure is protected by sheet piling.
	Review of survey data to date indicates no trends in structure movement.
	Piles support the Intake Structure, reducing the risk that the structure will be affected by shallow undermining.
	Survey data to date does not identify movement of the building.

Data Gaps:

- Observations of the riverbank following drawdown to normal river elevations
- Geophysical investigation data to address observed concerns
- Inclinometer readings that will provide an indication of slope movement

Conclusion

Significance

Potential for Degradation/Direct Floodwater Impact

River stage level has receded and stabilized at a level corresponding to the nominal "normal" river level at 40,000 cfs as of October 4, 2011. The potential for degradation from drawdown is low because it has not been observed as of October 4, 2011. Rapid drawdown has been controlled, and continued river drawdown is not expected to occur at a rate that would initiate this Triggering Mechanism. Because it is believed that a potential for degradation of the foundation exists but is not likely, the potential for degradation is considered low for this Triggering Mechanism and associated CPFMs 12a and 12b.

Implication

The occurrence of these CPFMs could lead to excessive movement and negatively impact the integrity or intended function of the structures and systems surrounding the Intake Structure. Therefore, the implication of the potential for degradation is high.

Confidence

At the time of the field report, conditions required to initiate the Triggering Mechanism associated with CPFMs 12a and 12b had not yet occurred. Field observations and other investigation data required to evaluate these CPFMs have not been made; therefore, an evaluation cannot be made.

The data available at the time of Revision 0 are not sufficient to rule out these CPFMs or lead to a recommendation for a physical modification to ensure that river bank slope failure and lateral spreading will not occur. Therefore, the confidence in the above assessment at this time is low.

Summary

For CPFMs 12a and 12b, as discussed above, the potential for degradation to the river bank surrounding the Intake Structure is low because the bank is protected. In the unlikely event that these CPFMs were to occur, the implication of this degradation to the structures and systems surrounding the Intake Structure is high. The combined consideration of the potential for degradation and the implications of that degradation to this structure put it in the "not significant" category. The data currently collected are not sufficient to rule out these CPFMs. Therefore, the confidence in the above assessment is low, which means that more data or continued monitoring and inspections are necessary to draw a conclusion. These data will be available in subsequent revisions of this Assessment Report.

5.1.5 Results and Conclusions

The CPFMs evaluated for the Intake Structure are presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**	CPFM 12a CPFM 12b	

5.1.6 Recommended Actions

Continued monitoring is recommended to include a continuation of the elevation surveys of the previously identified targets on this structure and surrounding site. In addition, a review of the ongoing geophysical investigations and monitoring of inclinometer readings is recommended. The purpose is to monitor for signs of structure distress and movement or changes in soil conditions around the structure. The results of this monitoring will be used to increase the confidence in the assessment results. Elevation surveys should be performed weekly for 4 weeks and biweekly until December 31, 2011. At the time of Revision 0, groundwater levels had not yet stabilized to nominal normal levels. Therefore, it is possible that new distress indicators could still develop. If new distress indicators are observed before December 31, 2011, appropriate HDR personnel should be notified immediately to determine if an immediate inspection or assessment should be conducted. Observation of new distress indicators might result in a modification of the recommendations for this structure.

5.1.7 Updates Since Revision 0

Revision 0 of this Assessment Report was submitted to OPPD on October 14, 2011. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure. These assessments were incomplete in Revision 0 because the forensic investigation and/or monitoring for most of the Priority 1 Structures was not completed by the submittal date. This revision of this Assessment Report includes the results of additional forensic investigation and monitoring to date for this structure as described below.

5.1.7.1 Additional Data Available

The following additional data were available for the Intake Structure for Revisions 1 and 2 of this Assessment Report:

- Additional groundwater monitoring well and river stage level data from OPPD.
- Field observations of the river bank (see Section 5.25).
- Results of geophysical investigation by Geotechnology, Inc. (see Attachment 6).
- Results of geotechnical investigation by Thiele Geotech, Inc. (see Attachment 6).
- Data obtained from inclinometers by Thiele Geotech, Inc. (see Attachment 6).
- Results of continued survey by Lamp Rynearson and Associates (see Attachment 6).

5.1.7.2 Additional Analysis

The following analysis of additional data was conducted for the Intake Structure:

- Groundwater monitoring well and river stage level data from OPPD.

  Data shows that the river and groundwater have returned to nominal normal levels.

- Field observations of river bank

  No significance distress from the 2011 Flood was observed.

- Results of geophysical investigation report by Geotechnology, Inc.

  Seismic Refraction and Seismic ReMi tests performed around the outside perimeter of the power block identified deep anomalies that could be gravel, soft clay, loose sand, or possibly voids.

- Results of geotechnical investigation by Thiele Geotech, Inc.

  Six test borings were drilled, with continuous sampling of the soil encountered, to ground truth the Geotechnology, Inc. seismic investigation results as part of the KDI #2 forensic investigation. Test bore holes were located to penetrate the deep anomalies identified in the seismic investigation. The test boring data did not show any piping voids or very soft/very loose conditions that might be indicative of subsurface erosion/piping or related material loss or movement.

  All of the SPT and CPT test results conducted for this Assessment Report were compared to similar data from numerous other geotechnical investigations that have been conducted on the FCS site in previous years. This comparison did not identify substantial changes to the soil strength and stiffness over that time period. SPT and CPT test results were not performed in the top 10 feet to protect existing utilities.

Data from inclinometers to date, compared to the original baseline measurements, have not exceeded the accuracy range of the inclinometers. Therefore, deformation at the monitored locations since the installation of the instrumentation has not occurred.

- Results of continued survey by Lamp Rynearson and Associates:

  Survey data to date compared to the original baseline surveys have not exceeded the accuracy range of the surveying equipment. Therefore, deformation at the monitored locations, since the survey baseline was shot, has not occurred.

The CPFMs that could not be ruled out in Revision 0 are analyzed below based on the additional data available for Revisions 1 and 2 of this Assessment Report.

Triggering Mechanism 12 – Rapid Drawdown
CPFM 12a – River bank slope failure and undermining surrounding structures
CPFM 12b – Lateral spreading

The Triggering Mechanism and CPFMs could occur as follows: The river level drops faster than pore water pressure in the soil can dissipate. The saturated soil is elevated above the dropping river level. The sloped bank of the river provides no lateral pressure support for the saturated soil. At some point, there is insufficient support on the river side to support the saturated soils. At that point, the soils experience slope movements or even failure. Generally, slope failures associated with rapid drawdown are relatively localized and shallow in nature; however, deeper failures can occur.

Significance

Potential for Degradation/Direct Floodwater Impact

The groundwater monitoring well data and river level data indicate that excess pore pressures due to river drawdown had generally dissipated by about October 14, 2011. Field observations of the River Bank on October 20, 2011, did not identify deformation of the river bank that could be attributed to slope failure or lateral spreading. Therefore, it can be concluded that neither slope failure nor lateral spreading occurred due to the 2011 flood.

Because it is believed that a potential for degradation of the structure exists but is not likely, the potential for degradation is considered low for this Triggering Mechanism and associated CPFMs 12a and 12b.

Implication

The occurrence of this potential degradation could lead to excessive movement and negatively impact the integrity or intended function of the structures and systems surrounding the Intake Structure. Therefore, the implication of the potential degradation for the Intake Structure is high.

Confidence

Field observations of the river bank and review of the groundwater data indicates that neither slope failure nor lateral spreading occurred due to the 2011 flood. Therefore, confidence in the results of this assessment for these CPFMs is high.

Summary

For CPFMs 12a and 12b, as discussed above, the potential for degradation to the river bank surrounding the Intake Structure is low because of field observations and analysis of groundwater data. In the unlikely event that these CPFMs were to occur, the implication of this degradation to the structures and systems surrounding the Intake Structure is high. The combined consideration of the potential for degradation and the implications of that degradation to this structure places it in the "not significant" category. The data collected since Revision 0 is sufficient to rule out these CPFMs assuming the previously recommended monitoring schedule is continued. Therefore, the confidence in the above assessment is high, which means no additional data and inspections are necessary to draw a conclusion. Assuming that no further concerns are identified through the monitoring program for the Intake Structure (discussed in Section 5.1.6 and continuing until December 31, 2011), these CPFMs are moved to the quadrant of the matrix representing "No Further Action Recommended Related to the 2011 Flood."

5.1.7.3 Revised Results

The CPFMs evaluated for the Intake Structure are presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**		CPFM 12a CPFM 12b

5.1.7.4 Conclusions

In the assessment of the FCS Structures, the first step was to develop a list of all Triggering Mechanisms and PFMs that could have occurred due to the prolonged inundation of the FCS site during the 2011 Missouri River flood and could have negatively impacted these structures. The next step was to use data from various investigations, including systematic observation of the structures over time, either to eliminate the Triggering Mechanisms and PFMs from the list or to recommend further investigation and/or physical modifications to remove them from the list for any particular structure. Because all CPFMs for the Intake Structure other than CPFMs 12a and 12b had been ruled out prior to Revision 1, and because CPFMs 12a and 12b have been ruled out as a result of the Revision 1 findings, no Triggering Mechanisms and their associated PFMs remain credible for the Intake Structure. Therefore, HDR has concluded that the 2011 Missouri River flood did not impact the geotechnical and structural integrity of the Intake Structure because the potential for failure of this structure due to the flood is not significant.

Section 5.2

Auxiliary Building

5.2 Auxiliary Building

5.2.1 Summary of Auxiliary Building

Baseline information for the Auxiliary Building is provided in Section 2.0, Site History, Description, and Baseline Condition.

The Auxiliary Building is located in the center of the PA. It is attached to the Turbine Building, Technical Support Building, CARP Building, and Rad Waste Building on its east, northeast, northwest, and west, respectively. The Auxiliary Building surrounds a majority of the Containment, which is located at its center.

The Auxiliary Building is a multi-story reinforced concrete Class I structure essential to plant operation. The building houses major systems and components, both CQE and non-CQE, in 81 designated rooms. The major function of the building is to provide the structural support and environmental protection necessary to ensure the functional integrity of the CQE systems and components under all operational and environmental conditions. The building also provides radiation shielding and mitigates radiological releases to the environment.

The Auxiliary Building basement floor elevation is approximately the same elevation as the basement slab for the Turbine Building (990 ft). The adjoining Technical Support Building, CARP Building, and Rad Waste Buildings are shallow-foundation structures with the ground level slabs roughly matching the ground level elevation of the Auxiliary Building (1007 ft). The basement floor elevation where it adjoins with the Technical Support Building and CARP Building is 989.00 ft. The basement floor elevation where it adjoins with the Rad Waste Building ranges from 971 to 989 ft. The grade-supported structures adjacent to the foundation walls apply a surcharge onto the foundation walls of the Auxiliary Building.

The Auxiliary Building is a reinforced concrete structure with a multitude of vertical concrete shear walls and concrete floor diaphragms that provide lateral support to the building. The building is supported on 20-in.-diameter open-ended steel piling extending into sound bedrock. Where solution cavities are located below the piles, the piles are underreamed and extended past the solution cavities. The piles are filled with sand up to 1 ft from the bottom of the mat, and the remainder of the piles are filled with concrete. The piles are capped with a 2-in.-thick ASTM A36 steel cap plate that is 22 in. by 22 in. The piles are spaced at approximately 9 ft on center. The pile-supported mat foundation varies in thickness from 12 ft to 1.5 ft, depending on the room use. The exterior walls are a minimum of 2.5-ft-thick reinforced concrete. The roof is cast-in-place reinforced concrete supported on concrete beams cast monolithically with the roof. Interior floors and walls are cast-in-place reinforced concrete. Thicknesses vary per room use and floor span.

The Containment, which resides at the center of the Auxiliary Building, is supported on 20-in.-diameter open-ended steel piling extending into sound bedrock. Where solution cavities are located below the piles, the piles are underreamed and extend past the solution cavities. The piles are filled with sand up to 1 ft from grade, and the remainder of the piles are filled with concrete. The piles are capped with a 2-in.-thick ASTM A36 steel cap plate that is 22 in. by 22 in. The number of piles in each circle is constant, with each pile circle 5 ft closer to the center. Therefore, the piling density increases as you reach the center of the Containment. The top of the foundation adjacent to the Auxiliary Building is at el. 991 ft. The top-of-foundation elevations for the Auxiliary Building, where it adjoins the Containment, range from 971 to 1002 ft.

5.2.2 Inputs/References Supporting the Analysis

Table 5.2-1 lists references provided by OPPD and other documents used to support HDR's analysis.

Table 5.2-1 - References for Auxiliary Building

Document Title	OPPD Document Number (if applicable)	Date	Page Number(s)
Incident Report Summary	CR 2011-5490	6/10/2011	All
Incident Report Summary	CR 2011-5560	6/14/2011	All
Incident Report Summary	CR 2011-5605	6/16/2011	All
Incident Report Summary	CR 2011-5609	6/16/2011	All
Incident Report Summary	CR 2011-5670	6/20/2011	All
Incident Report Summary	CR 2011-5837	6/28/2011	All
Incident Report Summary	CR 2011-5853	6/28/2011	All
Incident Report Summary	CR 2011-5961	7/4/2011	All
Incident Report Summary	CR 2011-5977	7/6/2011	All
Incident Report Summary	CR 2011-5978	7/6/2011	All
Incident Report Summary	CR 2011-6051	7/9/2011	All
Incident Report Summary	CR 2011-6051/6052	7/9/2011	All
System Design Basis Document	SDBD-AUX-502, Rev 18	1/19/2011	
Foundation Studies		3/19/1968	All
Soils Exploration Report		2/2/1967	All
Auxiliary Building Structural Inspection		5/22/1996	All
Naval Facilities Engineering Command, Design Manual 7.01, Soil Mechanics		9/1986	All

Detailed site observations—field reports, field notes, and inspection checklists—for the Auxiliary Building are provided in Attachment 8.

Observed performance and pertinent background data are as follows:

- A sand boil/piping feature was observed (originally reported in CR 2011-7265) near the southwest corner of the Missile Shield Room. This room is located on the outside of the south wall of the Auxiliary Building (common wall to both spaces) and has an unfinished, pea gravel floor surface. Ingress/egress can only be made from Door 45 located outside of the Auxiliary Building (there is no connecting doorway between the Missile Shield Room and the adjacent Auxiliary Building). This feature was measured using a hand tape measure and vertically probed using a 4-ft-long, 0.5-in.-diameter, steel-tipped fiberglass T-handle soil probe (commonly referred to as a foundation probe). Field measurements showed the feature was about 3.5 ft in diameter and 1 ft deep. A high-water line was observed on the interior walls approximately 0.8 ft above the floor. Various utility conduits extend vertically into the ground along the outside wall at the southwest corner (a few feet west of the alignment of the boil/piping feature). The Main Underground Cable Bank, MH-1 to the Auxiliary Building, passes through the subsurface, extending east to west below the location of the boil/piping feature.

- General observations of the interior of the structure showed minor concrete cracking in several walls and ceilings throughout the building. These hairline cracks could be associated with concrete shrinkage that is typically seen in concrete construction. The below-grade walls were dry at the time of inspection although there was evidence of past water infiltration. The observed cracks appear to be those previously recorded and monitored based on incident report summaries from OPPD listed in Table 5.2.1.
- The structure was protected from floodwaters for the majority of the 2011 flood by an Aqua Dam; however, the Aqua Dam failed for a short period of time due to being damaged, allowing floodwater to enter the area inside the Aqua Dam's perimeter. This incident resulted in the flooding of the truck dock room along the south elevation. Floodwaters reached approximately 2 ft in the dock room and infiltrated room 24A directly below the dock room through existing cracks in both the floor and walls.
- Voids were found below the slab in the Turbine Building, which is adjacent to and west of the Auxiliary Building. These voids were documented in 1997 to be approximately 1 ft deep. The extent of the voids is unknown, but they have been located approximately 15 ft from the wall shared with the Auxiliary Building. A constant flow of water through the voids at various rates since 1997 is believed to have occurred due to the constant sump pump operation in the Turbine Building. For further information see Section 5.8. A more detailed discussion of this Key Distress Indicator is presented in Section 4.1.
- The Maintenance Shop to the northeast has documented settlement issues. One building column footing had settled approximately 3 in. at the time of Revision 0. A section of floor slab on grade is settling. A more detailed discussion of this Key Distress Indicator is presented in Section 4.3.

5.2.3 Assessment Methods and Procedures

5.2.3.1 Assessment Procedures Accomplished

Assessments of the Auxiliary Building included the following:

- A visual inspection of the interior of the structure's lowest levels
- A visual inspection of the exterior of the structure where accessible
- A visual inspection of sumps in the Auxiliary Building for the presence of water, water level, and the presence of sediment in water
- An assessment of collected survey data to date for indications of trends in the movement of the structure
- A review of previously documented condition reports, as-built building plans and geotechnical reports to determine possible weak points in the building's construction that could be affected by the flood

Additional investigations were performed. These included the following non-invasive geophysical and invasive geotechnical investigations:

- Seismic surveys (seismic refraction and refraction micro-tremor) in the PA. (Test reports were not available at the time of Revision 0.)

- Geotechnical test borings in the protected area. Note that OPPD required vacuum excavation for the first 10 ft of proposed test holes to avoid utility conflicts. Therefore, test reports will not show soil conditions in the upper 10 ft of test boring logs. (Test reports were not available at the time of Revision 0.)
- Four inclinometers were installed on site to determine the condition of the riverbank by detecting lateral movement of the soils. (Inclinometers were not installed and thus no data were available at the time of Revision 0.)

5.2.3.2 Assessment Procedures Not Completed

No additional assessment procedures have been identified for this structure.

5.2.4 Analysis

Identified PFMs were initially reviewed as discussed in Section 3.0. The review considered the preliminary information available from OPPD data files and from initial walk-down observations. Eleven PFMs associated with five different Triggering Mechanisms were determined to be "non-credible" for all Priority 1 Structures, as discussed in Section 3.6. The remaining PFMs were carried forward as "credible." After the design review for each structure, the structure observations, and the results of available geotechnical, geophysical, and survey data were analyzed, a number of CPFMs were ruled out as discussed in Section 5.2.4.1. The CPFMs carried forward for detailed assessment are discussed in Section 5.2.4.2.

5.2.4.1 Potential Failure Modes Ruled Out Prior to the Completion of the Detailed Assessment

The ruled-out CPFMs reside in the Not Significant/High Confidence category and for clarity will not be shown in the Potential for Failure/Confidence matrix.

Triggering Mechanism 2 – Surface Erosion
CPFM 2b – Loss of lateral support for pile foundation

Reason for ruling out:

- It was evident from HDR's site observations that no surface erosion occurred in the vicinity of the Auxiliary Building.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3e – Loss of lateral support for pile foundation (due to river drawdown)

Reason for ruling out:

- The structure is a sufficient distance from the river to be outside the zone of influence of the CPFM.

Triggering Mechanism 4 – Hydrostatic Lateral Loading (water loading on structures)

CPFM 4c – Wall failure in flexure
CPFM 4d – Wall failure in shear
CPFM 4e – Excess deflection

Reasons for ruling out:

- The Auxiliary Building is designed to withstand an external water load due to flooding of the Missouri River to el. 1014 ft (see SDBD-AUX-502, Rev 18). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.
- Visual observations did not identify distress to the structure that can be attributed to this CPFM.

Triggering Mechanism 5 – Hydrodynamic Loading

CPFM 5a – Overturning
CPFM 5b – Sliding
CPFM 5c – Wall failure in flexure
CPFM 5d – Wall failure in shear
CPFM 5e – Damage by debris
CPFM 5f – Excess deflection

Reasons for ruling out:

- The structure was protected from floodwater by an Aqua Dam except during a short period of time when the Aqua Dam failed due to being damaged, which allowed floodwater to enter the area inside the Aqua Dam perimeter.
- The Auxiliary Building is designed to withstand an external water load due to flooding of the Missouri River to el. 1014 ft (see SDBD-AUX-502, Rev 18). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.
- Visual observation did not identify distress to the structure that can be attributed to this CPFM.

Triggering Mechanism 6 – Buoyancy, Uplift Forces on Structures

CPFM 6a – Fail tension piles
CPFM 6b – Cracked slab, loss of structural support
CPFM 6c – Displaced structure/broken connections

Reasons for ruling out:

- The Auxiliary Building is designed to withstand an external water load due to flooding of the Missouri River to el. 1014 ft (see SDBD-AUX-502, Rev 18). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.
- Visual observations and survey measurements show no structure movement. Therefore, it is unlikely that the tension piles failed (CPFM 6a) or that the structure was displaced or damaged (CPFM 6c) due to buoyancy effects.

Triggering Mechanism 7 – Soil Collapse (first time wetting)
CPFM 7b – Displaced structure/broken connections
CPFM 7c – General site settlement
CPFM 7d – Piles buckling from down drag

Reasons for ruling out:

- The pile foundations are located below el. 971.0 ft. while the normal river level is at approximate el. 992.0 ft. It is therefore logical to assume that the soils below the mat foundation were previously wetted.
- The peak flood elevation prior to 2011 was document in 1993 as 1003.3 ft, which would show that the soils below and surrounding the building were saturated at that time.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10b – Displaced structure/broken connections
CPFM 10c – Additional lateral force on below-grade walls
CPFM 10d – Pile/pile group instability

Reasons for ruling out:

- The underlying soils were improved with vibroflotation to reduce the risk of liquefaction.
- Temporary pumping equipment located on the ground within the Aqua Dam perimeter produced minimal localized vibration and was offset from the structure and therefore is deemed to have an inconsequential effect.
- Machine/vibration-induced liquefaction was not observed at the site.
- This is not a changed condition due to the flood. The Auxiliary Building has been in service for 38 years under similar saturated soils and machine vibration conditions.

Triggering Mechanism 11 – Loss of Soil Strength due to Static Liquefaction or Upward Seepage
CPFM 11b – Displaced structure/broken connections
CPFM 11c – Additional lateral force on below-grade walls
CPFM 11d – Pile/pile group instability

Reasons for ruling out:

- Visual observations and survey measurements show no structure movement. Therefore, degradation that can be attributed to this PFM did not occur.
- Sandboil/piping feature observed in the missile room was determined to be too shallow to be significant.
- The underlying soils were improved with vibroflotation to reduce the risk of liquefaction.

Triggering Mechanism 12 – Rapid Drawdown
CPFM 12a – River bank slope failure and undermining surrounding structures
CPFM 12b – Lateral spreading

Reason for ruling out:

- The Auxiliary Building is located a sufficient distance away from the river bank and therefore is outside the zone of influence of a bank slope failure.

Triggering Mechanism 13 – Submergence
CPFM 13b – Corrosion of structural elements

Reasons for ruling out:

- The Auxiliary Building has not been subjected to corrosive circumstances that would be considered beyond the normal conditions. The structure was protected from floodwater by an Aqua Dam except during a short period of time when the Aqua Dam failed due to being damaged, which allowed floodwater to enter the area inside the Aqua Dam perimeter.. Therefore, structural elements being wetted by the 2011 flood were considered in the original design of the facility.
- This is not a changed condition due to the flood. The Auxiliary Building has been in service for 38 years under similar saturated soils conditions. Reviewed condition survey reports have not indicated signs of distress to the structure that would be attributed to corrosion due to submergence.

Triggering Mechanism 14 – Frost Effects
CPFM 14a – Heaving, crushing, or displacement

Reason for ruling out:

- The Auxiliary Building foundation is approximately 25 ft below grade and therefore not frost susceptible. In addition, frost-susceptible connecting utilities are below the frost level.

5.2.4.2 Detailed Assessment of Credible Potential Failure Modes

The following CPFMs are the only CPFMs carried forward for detailed assessment for the Auxiliary Building as a result of the 2011 flood. This detailed assessment is provided below.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3b – Loss of lateral support for pile foundation (due to pumping)

The Turbine Building, which is connected to the Auxiliary Building on its east, has a documented history of a void below the foundation slab dating back to 1997. This void was confirmed via cored holes in the foundation slabs and camera recordings of broken drain piping that lies under the floor slab. Conversations with OPPD personnel indicate that groundwater has been flowing at varying rates through these broken pipes into the sump from that time to the present day. The rate of flow into the sump is directly related to the hydraulic head of the groundwater. As the floodwater increased in elevation across the site, observed flow rates increased. The flow of groundwater into this drain piping system through the breaks in the pipes is one of the Key Distress Indicators discussed in Section 4. This drain pipe system was designed as a closed system; therefore, the pipes are not surrounded by appropriate filter systems to preclude the transportation of soils from the surrounding area under the slab. It is logical to assume that because the groundwater moves below the foundation and into the broken piping, some movement of the soil has occurred. If these voids were to continue under the Auxiliary Building, they could become large enough to create a loss of lateral support for the piling.

The Triggering Mechanism and CPFM could then occur as follows: multiple potentially connected seepage paths could exist in the soil backfill at the site, including soil backfill in utility trenches, granular trench bedding, and building floor drains with open/broken joints. The paths could be exposed at some locations to the river floodwaters and high groundwater. This network of seepage paths could be connected to the sump pit in the Turbine Building. The breaks in the piping have been documented for an extended period (dating back to at least 1997), thus creating a continuous head differential on the potential seepage path networks. Gradient has been sufficient to begin erosion of surrounding soil. The gradient during the 2011 flood was increased, which could have led to higher flows through the seepage path networks. The unfiltered seepage condition will continue until the breaks in the piping system are repaired, which means the potential for further erosion remains. Erosion could create large voids under the Turbine Building base slab and potentially under adjacent building foundations, including the Auxiliary Building. The potential damage includes loss of soil support around piles leading to pile buckling, decreased pile capacity, and foundation failure.

The following table describes observed distress indicators and other data that would increase or decrease the potential for degradation associated with this CPFM for the Auxiliary Building.

Adverse (Degradation/Direct Floodwater Impact More Likely)	Favorable (Degradation/Direct Floodwater Impact Less Likely)
A documented void exists under the foundation slab of the Turbine Building with a known hydraulic connection between groundwater elevation and flows into the building sump.	The in-situ and fill material around the piling was compacted to the requirements under the Class I structures (vibroflotation). This higher density granular material is less susceptible to erosion.
	There have been no observed signs of structural distress in the floor slab under the current loading conditions.
	Survey data to date does not identify movement of the building.
Data Gaps: • The size and location of the voids below the foundation slab	

Conclusion

<u>Significance</u>

Potential for Degradation/Direct Floodwater Impact

Indicators for this CPFM have been observed in the Turbine Building, which is adjacent to the Auxiliary Building. The voids below the base slab in the Turbine Building are known to exist with heavy flows of water being pumped from the sump. Because the 2011 flood caused increased flow through the broken drain pipes, the potential that it caused further and more rapid degradation due to this CPFM is high. It is possible that these voids extend under the Auxiliary Building although the potential is low due to the vibro-compacted soils below the Auxiliary Building.

Implication

The occurrence of this CPFM on a large scale could negatively impact the capacity of the piling supporting the building. This could lead to excessive foundation movement and negatively impact the integrity or intended function of the Auxiliary Building. Therefore, the implication of the potential degradation for this CPFM is high.

Confidence

The extent of subsurface erosion and its potential impact on the building is not known due to the lack of data gathered on subsurface conditions. Because there is not enough information on the subsurface conditions at this time, and the pumping in the Turbine Building could have caused subsurface erosion, the confidence for this CPFM is low.

Summary

For CPFM 3b, as discussed, the potential for degradation is low due to the presence of vibrocompacted soils below the Auxiliary Building. This degradation could have caused enough erosion to impact the integrity or intended function of the structure. The combined consideration of the potential for degradation and the implications of that degradation to a structure of this type puts it in the "significant" category. The data currently collected are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is low, which means more data or continued monitoring and/or inspections are necessary to draw a conclusion.

5.2.5 Results and Conclusions

The CPFM evaluated for the Auxiliary Building is presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**	CPFM 3b	
Potential for Failure **Not Significant**		

5.2.6 Recommended Actions

Further forensic investigations and physical modifications are recommended to address CPFM 3b for the Auxiliary Building. CPFM 3b is associated with unfiltered flow of groundwater into the Turbine Building basement drain piping system (Key Distress Indicator #1). These recommendations are described in detail in Section 4.1.

Continued monitoring is recommended to include a continuation of the elevation surveys of the previously identified targets on this structure and surrounding site. The purpose is to monitor for signs of structure distress and movement or changes in soil conditions around the structure. The results of this monitoring will be used to increase the confidence in the assessment results. Elevation surveys should be performed weekly for 4 weeks and biweekly until December 31, 2011. At the time of Revision 0, groundwater levels had not yet stabilized to nominal normal levels. Therefore, it is possible that new distress indicators could still develop. If new distress indicators are observed before December 31, 2011, appropriate HDR personnel should be notified immediately to determine whether an immediate inspection or assessment should be conducted. Observation of new distress indicators might result in a modification of the recommendations for this structure.

5.2.7 Updates Since Revision 0

Revision 0 of this Assessment Report was submitted to OPPD on October 14, 2011. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure. These assessments were incomplete in Revision 0 because the forensic investigation and/or monitoring for most of the Priority 1 Structures was not completed by the submittal date. This revision of this Assessment Report includes the results of additional forensic investigation and monitoring to date for this structure as described below.

5.2.7.1 Additional Data Available

The following additional data were available for the Auxiliary Building for Revisions 1 and 2 of this Assessment Report:

- Results of KDI #1 forensic investigation (see Section 4.1).
- Additional groundwater monitoring well and river stage level data from OPPD.
- Results of geophysical investigation by Geotechnology, Inc. (see Attachment 6).
- Results of geotechnical investigation by Thiele Geotech, Inc. (see Attachment 6).
- Data obtained from inclinometers by Thiele Geotech, Inc. (see Attachment 6).
- Results of continued survey by Lamp Rynearson and Associates (see Attachment 6).

5.2.7.2 Additional Analysis

The following analysis of additional data was conducted for the Auxiliary Building:

- Groundwater monitoring well and river stage level data from OPPD.

  Data shows that the river and groundwater have returned to nominal normal levels.

- Results of geophysical investigation report by Geotechnology, Inc.

- Seismic Refraction and Seismic ReMi tests performed around the outside perimeter of the power block identified deep anomalies that could be gravel, soft clay, loose sand, or possibly voids.

- Results of geotechnical investigation by Thiele Geotech, Inc.

- Six test borings were drilled, with continuous sampling of the soil encountered, to ground truth the Geotechnology, Inc. seismic investigation results as part of the KDI #2 forensic investigation. Test bore holes were located to penetrate the deep anomalies identified in the seismic investigation. The test boring data did not show any piping voids or very soft/very loose conditions that might be indicative of subsurface erosion/piping or related material loss or movement.

  All of the SPT and CPT test results conducted for this Assessment Report were compared to similar data from numerous other geotechnical investigations that have been conducted on the FCS site in previous years. This comparison did not identify substantial changes to the soil strength and stiffness over that time period. SPT and CPT test results were not performed in the top 10 feet to protect existing utilities.

Data from inclinometers to date, compared to the original baseline measurements, have not exceeded the accuracy range of the inclinometers. Therefore, deformation at the monitored locations since the installation of the instrumentation has not occurred.

- Results of continued survey by Lamp Rynearson and Associates.

  Survey data to date compared to the original baseline surveys have not exceeded the accuracy range of the surveying equipment. Therefore, deformation at the monitored locations, since the survey baseline was shot, has not occurred.

- The CPFMs that could not be ruled out in Revision 0 are analyzed below based on the additional data available for Revisions 1 and 2 of this Assessment Report.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3b – Loss of lateral support for pile foundation (due to pumping)

CPFM 3b for the Auxiliary Building is associated with Key Distress Indicator #1. Section 4.1 presents the results of the additional forensic investigation that was conducted to ascertain whether this CPFM could be ruled out. The results of the additional forensic investigations show that if the recommendations for physical modifications in KDI #1 are implemented that this CPFM is ruled out. Therefore, assuming that no further concerns are identified through the monitoring program for the Auxiliary Building (discussed in Section 5.2.6 and continuing until December 31, 2011), this CPFM is moved to the quadrant of the matrix representing "No Further Action Recommended Related to the 2011 Flood."

5.2.7.3 Revised Results

The CPFM evaluated for the Auxiliary Building is presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**		CPFM 3b

5.2.7.4 Conclusions

In the assessment of the FCS Structures, the first step was to develop a list of all Triggering Mechanisms and PFMs that could have occurred due to the prolonged inundation of the FCS site during the 2011 Missouri River flood and could have negatively impacted these structures. The next step was to use data from various investigations, including systematic observation of the structures over time, either to eliminate the Triggering Mechanisms and PFMs from the list or to recommend further investigation and/or physical modifications to remove them from the list for any particular structure. Because all CPFMs for the Auxiliary Building other than CPFM 3b had been ruled out prior to Revision 1, and because CPFM 3b will be ruled out when the physical modifications recommended for KDI #1 in Section 4.1 are implemented, no Triggering Mechanisms and their associated PFMs will remain credible for the Auxiliary Building. HDR has concluded that the geotechnical and structural impacts of the 2011 Missouri River flood will be mitigated by the implementation of the physical modifications recommended in this Assessment Report. Therefore, after the implementation of the recommended physical modifications, the potential for failure of this structure due to the flood will not be significant.

Section 5.3

Containment

5.3 Containment

5.3.1 Summary of Containment

Baseline information for the Containment is provided in Section 2.0, Site History, Description, and Baseline Condition.

The Containment is surrounded by the Auxiliary Building on the north, east, and west sides, and on a portion of the south side. The Containment basement top-of-slab elevation is 976.5 ft below the reactor and 991 ft along the perimeter. A tunnel is located around the perimeter below the slab to access the post-tension cables (stressing gallery). The floor elevation of the stressing gallery is about 969 ft. The basement floor elevation of the Auxiliary Building where the Containment adjoins it ranges from el. 971 to 1004 ft. The outside grade is approximately at el. 1004 ft.

The Containment is supported on 20-in.-diameter open-ended steel pilings with 1.031-in.-thick walls extending into sound bedrock. Where solution cavities were located below the piles, they were underreamed and extended past the cavities. The piles are filled with sand up to 1 ft from grade, and the remainder is filled with concrete. To provide a solid bearing surface, the piles are capped with a 2-in.-thick ASTM A36 steel cap plate that is 22 in. by 22 in. The number of piles in each circle is constant, with each pile circle 5 ft closer to the center. Therefore, the piling density increases toward the center of the building. The stressing gallery tunnel is not supported by piles.

The mat foundation is a 10- to 12-ft-thick concrete slab reinforced with two layers of mild reinforcing. The exterior walls are approximately 3.9-ft-thick post-tensioned reinforced concrete. A steel liner is located on the interior of the wall. The roof is a 55-ft-radius concrete dome monolithically integrated into the exterior walls. Interior floors and walls are cast-in-place reinforced concrete. Thicknesses vary per room use and floor span.

5.3.2 Inputs/References Supporting the Analysis

Table 5.3-1 lists references provided by OPPD and other documents used to support HDR's analysis.

Table 5.3-1 - References for Containment

Document Title	OPPD Document Number (if applicable)	Date	Page Number(s)
Condition Report	CR 2011-5761	6/23/2011	All
Condition Report	CR 2011-5763	6/23/2011	All
Condition Report	CR 2011-5792	6/24/2011	All
Condition Report	CR 2011-7265	9/9/2011	All
System Design Basis Document	SDBD-CONT-501, Rev 32	9/30/2010	
Piling Plan Containment & Auxiliary Building	11405-S-1 (#16380)	5/6/1968	
Structure Inspection	SE-PM-AE-1004	7/16/2009	All
Naval Facilities Engineering Command, Design Manual 7.01, Soil Mechanics		9/1986	All

Detailed site observations—field reports, field notes, and inspection checklists—for the Containment are provided in Attachment 8.

Observed performance and pertinent background data are as follows:

- A sand boil/piping feature was observed (originally reported in CR 2011-7265) near the southwest corner of the Missile Shield Room. This room is located on the outside of the south wall of the Auxiliary Building (common wall to both spaces) and has an unfinished, pea gravel floor surface. Ingress/egress can be gained only from Door 45, located outside the Auxiliary Building (there is no connecting doorway between the Missile Shield Room and the adjacent Auxiliary Building). This feature was measured using a hand tape measure and vertically probed using a 4-ft-long, 0.5-in.-diameter, steel-tipped fiberglass T-handle soil probe (commonly referred to as a foundation probe). Field measurements indicated that the feature is about 3.5 ft in diameter and 1 ft deep. A high-water line was observed on the interior walls about 0.8 ft above the floor. Various utility conduits extend vertically into the ground along the outside wall at the southwest corner (a few feet west of the alignment of the boil/piping feature). The Main Underground Cable Bank, MH-1 to the Auxiliary Building, runs through the subsurface, extending east to west below the location of the boil/piping feature.
- Voids were found below the slab in the Turbine Building, which is adjacent to and west of the Auxiliary Building. These voids were documented in 1997 to be approximately 1 ft deep. The extent of the voids is unknown, but they have been located approximately 15 ft from the wall shared by the Auxiliary Building and the Turbine Building. A constant flow of water through the voids at various rates since 1997 is believed to have occurred due to the constant sump pump operation in the Turbine Building. For further information see Section 5.8.
- The stressing gallery is a tunnel located below the mat floor slab of the Containment. The gallery has one entrance in and out from Room 22 of the Auxiliary Building. The gallery provides access to the Containment wall post-tensioning strands and runs the entire perimeter of the Containment. The stressing gallery was found to contain a large amount of water in low level areas of the floor near the two sump pits. Water covered approximately half of the floor area at the time of HDR's inspection. Water was approximately 4 in. deep at the sump pit locations and decreased in depth away from the pits due to floor slab slope. The source of the water was not apparent from the inspection. Previous testing of the water by OPPD discovered that the water contained Cesium-134. Very little sediment was seen in the water.
- Pumps had been removed from the sumps in the stressing gallery prior to HDR's inspection.
- The structure was protected from floodwater for the majority of the 2011 flood by an Aqua Dam; however, the Aqua Dam failed for a short period of time due to being damaged, allowing floodwater to enter the area inside the Aqua Dam perimeter.

5.3.3 Assessment Methods and Procedures

5.3.3.1 Assessment Procedures Accomplished

Assessments of the Containment included the following:

- Visual inspection of the interior of the Containment's lowest levels (perimeter rooms only). Visual inspection of other interior rooms is not necessary to provide report results.
- Visual inspection of the exposed, above-grade exterior of the structure.
- Visual inspection of sumps in the stressing gallery for the presence of water, the water level, and the presence of sediment in the water.

- An assessment of collected survey data to date for indications of trends in the movement of the structure.
- A review of previously documented condition reports, as-built building plans, and geotechnical reports to determine possible weak points in the Containment's construction that could be affected by the 2011 flood.

Additional investigations were performed. These included the following non-invasive geophysical and invasive geotechnical investigations:

- Seismic surveys (seismic refraction and refraction micro-tremor) in the PA. (Test reports were not available at the time of Revision 0.)
- Geotechnical test borings in the PA. Note that OPPD required vacuum excavation for the first 10 ft of proposed test holes to avoid utility conflicts. Therefore, test reports will not show soil conditions in the upper 10 ft of test boring logs. (Test reports were not available at the time of Revision 0.)

5.3.3.2 Assessment Procedures Not Completed

No additional assessment procedures have been identified for this structure.

5.3.4 Analysis

Identified PFMs were initially reviewed as discussed in Section 3.0. The review considered the preliminary information available from OPPD data files and from initial walk-down observations. Eleven PFMs associated with five different Triggering Mechanisms were determined to be "non-credible" for all Priority 1 Structures, as discussed in Section 3.6. The remaining PFMs were carried forward as "credible." After the design review for each structure, the structure observations, and the results of available geotechnical, geophysical, and survey data were analyzed, a number of CPFMs were ruled out as discussed in Section 5.3.4.1. The CPFMs carried forward for detailed assessment are discussed in Section 5.3.4.2.

5.3.4.1 Potential Failure Modes Ruled Out Prior to the Completion of the Detailed Assessment

The ruled-out CPFMs reside in the Not Significant/High Confidence category and for clarity will not be shown in the Potential for Failure/Confidence matrix.

Triggering Mechanism 2 – Surface Erosion
CPFM 2b – Loss of lateral support for pile foundation

Reason for ruling out:

- It was evident from the site inspection that no surface erosion occurred in the vicinity of the Containment.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3e – Loss of lateral support for pile foundation (due to river drawdown)

Reason for ruling out:

- The structure is a sufficient distance from the river to be outside the zone of influence of the CPFM.

Triggering Mechanism 4 – Hydrostatic Lateral Loading (water loading on structures)
CPFM 4c – Wall failure in flexure
CPFM 4d – Wall failure in shear
CPFM 4e – Excess deflection

Reasons for ruling out:

- The Containment is designed to withstand an external water load due to flooding of the Missouri River to el. 1014 ft (see SDBD-CONT-501, Rev 32). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.
- Visual observations did not identify distress to the Containment that can be attributed to this CPFM.

Triggering Mechanism 5 – Hydrodynamic Loading
CPFM 5a – Overturning
CPFM 5b – Sliding
CPFM 5c – Wall failure in flexure
CPFM 5d – Wall failure in shear
CPFM 5e – Damage by debris
CPFM 5f – Excess deflection

Reasons for ruling out:

- The structure was protected from floodwater by an Aqua Dam except during a short period of time when the Aqua Dam failed due to being damaged, which allowed floodwater to enter the area inside the Aqua Dam perimeter..
- The Containment is designed to withstand an external water load due to flooding of the Missouri River to el. 1014 ft (see SDBD-CONT-501, Rev 32). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.
- Visual observations did not identify distress to the structure that can be attributed to this CPFM.

Triggering Mechanism 6 – Buoyancy, Uplift Forces on Structures
CPFM 6a – Fail tension piles
CPFM 6b – Cracked slab, loss of structural support
CPFM 6c – Displaced structure/broken connections

Reasons for ruling out:

- The Containment is designed to withstand an external water load due to flooding of the Missouri River to el. 1014 ft (see SDBD-CONT-501, Rev 32). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.

- Visual observations and survey measurements indicate no structure movement. Therefore, it is unlikely that the tension piles failed (CPFM 6a) or that the structure was displaced or damaged (CPFM 6c) due to buoyancy effects.

Triggering Mechanism 7 – Soil Collapse (first time wetting)
CPFM 7b – Displaced structure/broken connections
CPFM 7c – General site settlement
CPFM 7d – Piles buckling from down drag

Reasons for ruling out:

- The pile foundations are located below el. 979.0 ft, while the normal river level is at approximate el. 992.0 ft. Therefore it is logical to assume that the soils below the mat foundation have been previously wetted.
- The peak flood elevation prior to 2011 was documented as 1003.3 ft, which would indicate the soils below and surrounding the Containment had been saturated at this time.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10b – Displaced structure/broken connections
CPFM 10c – Additional lateral force on below-grade walls
CPFM 10d – Pile/pile group instability

Reasons for ruling out:

- The underlying soils were improved with vibroflotation to reduce the risk of liquefaction.
- Machine/vibration-induced liquefaction was not observed at the site.
- This is not a changed condition due to the flood. The Containment has been in service for 38 years under similar saturated soils and machine vibration.
- Temporary pumping equipment located on the ground within the Aqua Dam perimeter produced minimal localized vibrations and was offset from the structure and therefore is deemed to have inconsequential effects.

Triggering Mechanism 11 – Loss of Soil Strength due to Static Liquefaction or Upward Seepage
CPFM 11b – Displaced structure/broken connections
CPFM 11c – Additional lateral force on below-grade walls
CPFM 11d – Pile/pile group instability

Reasons for ruling out:

- The underlying soils were improved with vibroflotation to reduce the risk of liquefaction.
- The sandboil/piping feature observed in the missile room of the Auxiliary Building was determined to be too shallow to be significant.
- Visual observations and survey measurements indicate no structure movement. Therefore, degradation that can be attributed to this PFM did not occur.

Triggering Mechanism 12 – Rapid Drawdown
CPFM 12a – River bank slope failure and undermining surrounding structures
CPFM 12b – Lateral spreading

Reason for ruling out:

- The Containment is located a sufficient distance away from the riverbank and therefore is outside the zone of influence of a bank slope failure.

Triggering Mechanism 13 – Submergence
CPFM 13b – Corrosion of structural elements

Reasons for ruling out:

- The Containment has not been subjected to corrosive circumstances that would be considered beyond the normal conditions. The structure was protected from floodwater by an Aqua Dam except during a short period of time when the Aqua Dam failed due to being damaged, which allowed floodwater to enter the area inside the Aqua Dam perimeter. Therefore, any structural elements being wetted by the 2011 flood was considered in the original design of the facility.
- This is not a changed condition due to the flood. The Containment has been in service for 38 years under similar saturated soils. Reviewed condition survey reports have not indicated signs of distress that would be attributed to corrosion due to submergence.

Triggering Mechanism 14 – Frost Effects
CPFM 14a – Heaving, crushing, or displacement

Reason for ruling out:

- The Containment foundation is a minimum of 25 ft below grade and is therefore not susceptible to frost. In addition, frost-susceptible connecting utilities are also below frost level.

5.3.4.2 Detailed Assessment of Credible Potential Failure Modes

The following CPFMs are the only CPFMs carried forward for detailed assessment for the Containment as a result of the 2011 flood. This detailed assessment is provided below.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3b – Loss of lateral support for pile foundation (due to pumping)

The Turbine Building, which is adjacent to the Containment, has a documented history of a void below the foundation slab dating back to 1997. This void was confirmed via cored holes in the foundation slabs and camera recordings of broken drain piping that lies under the floor slab. Conversations with OPPD personnel indicate that groundwater has been flowing at varying rates through these broken pipes into the sump from that time to the present day. The rate of flow into the sump is directly related to the hydraulic head of the groundwater. As the floodwater increased in elevation across the facility, observed flow rates increased. The flow of groundwater into this drain piping system through the breaks in the pipes is one of the Key Distress Indicators discussed in Section 4. This drain pipe system was designed as a closed system; therefore, the pipes are not surrounded by appropriate filter systems to preclude the transportation of soils from the surrounding area under the slab. It is possible to assume that because the groundwater moves below the foundation and into the broken piping, some movement of the soil has occurred. If these voids were to continue under the Containment, they could become large enough to create a loss of lateral support for the piling.

The Triggering Mechanism and CPFM could then occur as follows: multiple potentially connected seepage paths could exist in the soil backfill at the site, including soil backfill in utility trenches, granular trench bedding, and building floor drains with open/broken joints. The paths could be exposed at some locations to the river floodwaters and high groundwater. This network of seepage paths could be connected to the sump pit in the Turbine Building. The breaks in the piping have been documented for an extended period (dating back to at least 1997), thus creating a continuous head differential on the potential seepage path networks. Gradient was potentially sufficient to begin erosion of surrounding soil. The gradient during the 2011 flood was increased, which could have led to higher flows through the seepage path networks. The unfiltered seepage condition will remain until the breaks in the piping system are repaired, which means the potential for further erosion remains. Erosion could extend out, creating large voids under the Turbine Building base slab and potentially under the Containment. The potential damage includes loss of soil support around piles leading to pile buckling, decreased pile capacity, and foundation failure.

The following table describes observed distress indicators and other data that would increase or decrease the potential for degradation associated with this CPFM for the Containment.

Adverse (Degradation/Direct Floodwater Impact More Likely)	Favorable (Degradation/Direct Floodwater Impact Less Likely)
A documented void exists under the foundation slab of the Turbine Building with a known hydraulic connection between groundwater elevation and flows into the building sump.	The in situ and fill material around the piling was compacted to the requirements under the Class I structures (vibroflotation). This higher density granular material is less susceptible to erosion.
	There have been no observed signs of structural distress in the floor slab under the current loading conditions.
	Surveyed elevations for the foundations show no significant signs of movement.
	The bottom of the mat foundation is about 10 ft lower in elevation than the bottom of the Turbine Building mat foundation, making it unlikely that voids migrated below the Containment foundation.

Data Gaps:

- The presence, size, and location of the voids below the foundation slab

Conclusion

Significance

Potential for Degradation/Direct Floodwater Impact

Indicators for this CPFM have been observed in the Turbine Building, which is near the Containment. The voids below the base slab in the Turbine Building are known to exist with heavy flows of water being pumped from the sump. Because the 2011 flood caused increased flow through the broken drain pipes, the potential that the 2011 flood caused further and more rapid degradation due to this CPFM is high. It is possible, but not likely, that these voids extend under the Auxiliary Building and to the Containment mat foundation. The potential for degradation is low due to the distance between the Turbine Building and the Containment and the presence of vibrocompacted soils under both the Auxiliary Building and Containment.

Implication

The occurrence of this CPFM on a large scale could negatively impact the capacity of the piling supporting the building. This could lead to excessive foundation movement and negatively impact the integrity or intended function of the Containment. Because the pile system is robust, and voids of this size are not likely, the implication of the potential degradation for this CPFM is low.

Confidence

The extent of subsurface erosion and its potential impact on the building is not known due to the lack of data gathered on subsurface conditions. Because there is not enough information on the subsurface conditions at this time, and the pumping in the Turbine Building could have caused subsurface erosion, the confidence for this CPFM is low.

Summary

For CPFM 3b, as discussed above, the potential for degradation is low because the pumping in the Turbine Building is unlikely to have caused enough erosion to impact the integrity or intended function of the structure. Although large amounts of erosion are not likely, large depths of erosion and degradation could impact the integrity or intended function of the structure. The combined consideration of the potential for degradation and the implications of that degradation to a structure of this type puts it in the "not significant" category. The data currently collected are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is low, which means more data and/or continued monitoring and inspections are necessary to draw a conclusion.

5.3.5 Results and Conclusions

The CPFM evaluated for the Containment is presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**	CPFM 3b	

5.3.6 Recommended Actions

Further forensic investigations and physical modifications are recommended to address CPFM 3b for the Containment. CPFM 3b is associated with unfiltered flow of groundwater into the Turbine Building basement drain piping system (Key Distress Indicator #1). These recommendations are described in detail in Section 4.1.

Water observed in the pre-stressing gallery cannot be attributed to a specific source at this time. To determine the water's source, it is suggested that the water be removed and a procedure developed and implemented to determine the source of the water. Once a source is determined, the proper personnel should be notified and the area inspected to determine whether further analysis or corrections are necessary.

Continued monitoring is recommended to include a continuation of the elevation surveys of the previously identified targets on this structure and surrounding site. The purpose is to monitor for signs of structure distress and movement or changes in soil conditions around the structure. The results of this monitoring increase the confidence in the assessment results. Elevation surveys should be performed weekly for 4 weeks and biweekly until December 31, 2011. At the time of Revision 0, groundwater levels had not yet stabilized to nominal normal levels. Therefore, it is possible that new distress indicators could still develop. If any new distress indicators are observed before December 31, 2011, appropriate HDR personnel should be notified immediately to determine whether an immediate inspection or assessment should be conducted. Observation of any new distress indicators might result in a modification of the recommendations for this structure.

5.3.7 Updates Since Revision 0

Revision 0 of this Assessment Report was submitted to OPPD on October 14, 2011. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure. These assessments were incomplete in Revision 0 because the forensic investigation and/or monitoring for most of the Priority 1 Structures was not completed by the submittal date. This revision of this Assessment Report includes the results of additional forensic investigation and monitoring to date for this structure as described below.

5.3.7.1 Additional Data Available

The following additional data were available for the Containment for Revisions 1 and 2 of this Assessment Report:

- Results of KDI #1 forensic investigation (see Section 4.1)
- Results of geophysical investigation by Geotechnology, Inc. (see Attachment 6).
- Results of geotechnical investigation by Thiele Geotech, Inc. (see Attachment 6).
- Data obtained from inclinometers by Thiele Geotech, Inc. (see Attachment 6).
- Results of continued survey by Lamp Rynearson and Associates (see Attachment 6).

5.3.7.2 Additional Analysis

The following analysis of additional data was conducted for the Containment:

- Results of geophysical investigation report by Geotechnology, Inc.

  Seismic Refraction and Seismic ReMi tests performed around the outside perimeter of the power block identified deep anomalies that could be gravel, soft clay, loose sand, or possibly voids.

- Results of geotechnical investigation by Thiele Geotech, Inc.

  Six test borings were drilled, with continuous sampling of the soil encountered, to ground truth the Geotechnology, Inc. seismic investigation results as part of the KDI #2 forensic investigation. Test boreholes were located to penetrate the deep anomalies identified in the seismic investigation. The test boring data did not show any piping voids or very soft/very loose conditions that might be indicative of subsurface erosion/piping or related material loss or movement.

  All of the SPT and CPT test results conducted for this Assessment Report were compared to similar data from numerous other geotechnical investigations that have been conducted on the FCS site in previous years. This comparison did not identify substantial changes to the soil strength and stiffness over that time period. SPT and CPT test results were not performed in the top 10 feet to protect existing utilities.

Data from inclinometers to date, compared to the original baseline measurements, have not exceeded the accuracy range of the inclinometers. Therefore, deformation at the monitored locations since the installation of the instrumentation has not occurred.

- Results of continued survey by Lamp Rynearson and Associates.

  Survey data to date compared to the original baseline surveys have not exceeded the accuracy range of the surveying equipment. Therefore, deformation at the monitored locations, since the survey baseline was shot, has not occurred.

The CPFMs that could not be ruled out in Revision 0 are analyzed below based on the additional data available for Revisions 1 and 2 of this Assessment Report.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3b – Loss of lateral support for pile foundation (due to pumping)

CPFM 3b for the Containment is associated with Key Distress Indicator #1. Section 4.1 presents the results of additional forensic investigation that was conducted to ascertain whether this CPFM could be ruled out. The results of the additional forensic investigations show that if the recommendations for physical modifications in KDI #1 are implemented that this CPFM is ruled out. Therefore, assuming that no further concerns are identified through the monitoring program for the Containment (discussed in Section 5.3.6 and continuing until December 31, 2011), this CPFM is moved to the quadrant of the matrix representing "No Further Action Recommended Related to the 2011 Flood."

5.3.7.1 Revised Results

The CPFM evaluated for the Containment is presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**		CPFM 3b

5.3.7.2 Conclusions

In the assessment of the FCS Structures, the first step was to develop a list of all Triggering Mechanisms and PFMs that could have occurred due to the prolonged inundation of the FCS site during the 2011 Missouri River flood and could have negatively impacted these structures. The next step was to use data from various investigations, including systematic observation of the structures over time, either to eliminate the Triggering Mechanisms and PFMs from the list or to recommend further investigation and/or physical modifications to remove them from the list for any particular structure. Because all CPFMs for the Containment other than CPFM 3b had been ruled out prior to Revision 1, and because CPFM 3b will be ruled out when the physical modifications recommended for KDI #1 in Section 4.1 are implemented, no Triggering Mechanisms and their associated PFMs will remain credible for the Containment. HDR has concluded that the geotechnical and structural impacts of the 2011 Missouri River flood will be mitigated by the implementation of the physical modifications recommended in this Assessment Report. Therefore, after the implementation of the recommended physical modifications, the potential for failure of this structure due to the flood will not be significant.

Section 5.4
Rad Waste Building

5.4 Rad Waste Building

5.4.1 Summary of Rad Waste Building

Baseline information for the Rad Waste Building is provided in Section 2.0, Site History, Description, and Baseline Condition.

The Rad Waste Building is a single-story, rectangular-shaped building with plan dimensions of about 73 by 175 ft. The building was added onto the south side of the Auxiliary Building. The roof is supported by a steel moment frame that transfers the load to a structural mat foundation. Exterior walls consist of precast panels, and interior walls consist of masonry block. The top of the mat foundation ranges from about el. 1002 to 1007 ft and is thickest along the exposed perimeter where it extends to 4 ft below exterior grades (up to 7 ft thick). The basement floor elevation of the Auxiliary Building where the Rad Waste Building abuts is 987 ft. Excavations for the Auxiliary Building would have extended to about el. 983.5 ft. Site grades prior to the original development ranged from about el. 1002 to 1004 ft. This would suggest placement of about 20 ft of backfill along the Auxiliary Building wall and minimal placement of additional fill to establish design grades. Excavations as deep as about 4 ft would have been necessary where the top of the mat foundation is established at 1002 ft.

5.4.2 Inputs/References Supporting the Analysis

Table 5.4-1 lists references provided by OPPD and other documents used to support HDR's analysis.

Table 5.4-1 - References for Rad Waste Building

Document Title	OPPD Document Number (if applicable)	Date	Page Number(s)
Foundation Plan North Area	7753-03-A-12 (#46694)	10/03/1988	
Foundation Plan South Area	7753-03-A-13 (#46694)	10/03/1988	
(Site Topo)		Unknown	
Bathymetric Survey		Unknown	
Survey Point Elevations		Unknown	
Naval Facilities Engineering Command Design Manual 7.01, Soil Mechanics		9/1986	All

Detailed site observations—field reports, field notes, and inspection checklists—for the Rad Waste Building are provided in Attachment 8.

Observed performance and pertinent background data are as follows:

- Vibrofloation was not documented to have been performed below the structure.
- The electrical ductbank located inside the Rad Waste Building was present prior to construction.
- The sump in the truck bay did not have groundwater infiltration at the time of the field assessment.
- The structure was protected from floodwater for the majority of the 2011 flood by an Aqua Dam; however, the Aqua Dam failed for a short period of time due to being damaged, allowing floodwater to enter the area inside the Aqua Dam perimeter. This incident resulted in the flooding of the truck bay. Floodwater flowed into the sump until the temporary flood barrier was installed.

- Water that is pumped from the sumps is documented and stored.
- Hairline and stair-step cracking was observed in the masonry walls at various locations. It is unclear whether these cracks were present prior to the 2011 flood.
- The structure is designated as a Class I (seismic) structure in accordance with OPPD.
- No incident report summaries or inspection records are available for this structure.
- No design basis summary document is available for this structure.
- General observations of the interior of the structure were limited by the accessibility in certain rooms.

5.4.3 Assessment Methods and Procedures

5.4.3.1 Assessment Procedures Accomplished

Assessments of the Rad Waste Building included the following:

- Visual inspection of the accessible areas of the interior of the structure
- Visual inspection of the accessible exterior of the structure
- An assessment of collected survey data to date for indications of trends in the movement of the structure
- Review of previously documented condition reports, as-built plans, site topography, and geotechnical reports to identify possible conditions that could be affected by the 2011 flood

Additional investigations were performed. These included the following non-invasive geophysical and invasive geotechnical investigations:

- Seismic surveys (seismic refraction and seismic ReMi) in the protected area. (Test reports were not available at the time of Revision 0.)
- Geotechnical test borings in the protected area. Note that OPPD required vacuum excavation for the first 10 ft of proposed test holes to avoid utility conflicts. Therefore, test reports will not show soil conditions in the upper 10 ft of test boring logs. (Test reports were not available at the time of Revision 0.)

5.4.3.2 Assessment Procedures Not Completed

No additional assessment procedures have been identified for this structure.

5.4.4 Analysis

Identified PFMs were initially reviewed as discussed in Section 3.0. The review considered the preliminary information available from OPPD data files and from initial walk-down observations. Eleven PFMs associated with five different Triggering Mechanisms were determined to be "non-credible" for all Priority 1 Structures, as discussed in Section 3.6. The remaining PFMs were carried forward as "credible." After the design review for each structure, the structure observations, and the results of available geotechnical, geophysical, and survey data were analyzed, a number of CPFMs were ruled out as discussed in Section 5.4.4.1. The CPFMs carried forward for detailed assessment are discussed in Section 5.4.4.2.

5.4.4.1 Potential Failure Modes Ruled Out Prior to the Completion of the Detailed Assessment

The ruled-out CPFMs reside in the Not Significant/High Confidence category and for clarity will not be shown in the Potential for Failure/Confidence matrix.

Triggering Mechanism 2 – Surface Erosion
CPFM 2a – Undermining shallow foundation/slab/surfaces

Reasons for ruling out:

- The structure was protected from the floodwater by an Aqua Dam except during a short period of time when the Aqua Dam failed due to being damaged, which allowed floodwater to enter the area inside the Aqua Dam perimeter.
- Surface erosion was not identified near the structure during the field assessments.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3a – Undermining and settlement of shallow foundation/slab/surfaces (due to pumping)

Reason for ruling out:

- The structure is a sufficient distance from the known pumping locations to be outside the zone of influence of the CPFM.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3d – Undermining and settlement of shallow foundation/slab (due to river drawdown)

Reason for ruling out:

- The structure is a sufficient distance from the river to be outside the zone of influence of the CPFM.

Triggering Mechanism 4 – Hydrostatic Lateral Loading (water loading on structures)
CPFM 4a – Overturning
CPFM 4b – Sliding
CPFM 4c – Wall failure in flexure
CPFM 4d – Wall failure in shear
CPFM 4e – Excess deflection

Reasons for ruling out:

- The Rad Waste Building is designed to withstand an external water load due to flooding of the Missouri River to el. 1007 ft. The peak flood elevation in 2011 was approximately 1006.9 ft.
- Visual observation did not identify distress to the structure that can be attributed to this CPFM.

Triggering Mechanism 5 – Hydrodynamic Loading

CPFM 5a – Overturning
CPFM 5b – Sliding
CPFM 5c – Wall failure in flexure
CPFM 5d – Wall failure in shear
CPFM 5e – Damage by debris
CPFM 5f – Excess deflection

Reasons for ruling out:

- Sufficient high floodwater velocities were not identified near the structure.
- The structure was protected from the floodwater by an Aqua Dam except during a short period of time when the Aqua Dam failed due to being damaged, which allowed floodwater to enter the area inside the Aqua Dam perimeter.
- Visual observation did not identify distress to the structure that can be attributed to this CPFM.

Triggering Mechanism 6 – Buoyancy, Uplift Forces on Structures

CPFM 6b – Cracked slab, loss of structural support
CPFM 6c – Displaced structure/broken connections

Reason for ruling out:

- The Rad Waste Building is designed to withstand an external water load due to flooding of the Missouri River to el. 1007 ft. The peak flood elevation in 2011 was approximately 1006.9 ft.

Triggering Mechanism 7 – Soil Collapse (first time wetting)

CPFM 7c – General site settlement

Reasons for ruling out:

- Site settlement was not observed near the Rad Waste Building during the field assessments.
- The peak flood elevation prior to 2011 was documented in 1993 at 1003.3 ft, which would indicate the soils below and surrounding the building have been saturated.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction

CPFM 10a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 10b – Displaced structure/broken connections
CPFM 10c – Additional lateral force on below-grade walls

Reason for ruling out:

- Machine/vibration-induced liquefaction was not observed to have occurred at the site.

Triggering Mechanism 11 – Loss of Soil Strength due to Static Liquefaction or Upward Seepage

CPFM 11a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 11b – Displaced structure/broken connections
CPFM 11c – Additional lateral force on below-grade walls

Reason for ruling out:

- Machine/vibration-induced liquefaction was not observed to have occurred at the site.

Triggering Mechanism 12 – Rapid Drawdown

CPFM 12a – River bank slope failure and undermining surrounding structures
CPFM 12b – Lateral spreading

Reason for ruling out:

- The Rad Waste Building is a sufficient distance from the river to be outside the zone of influence of the CPFM.

Triggering Mechanism 14 – Frost Effects

CPFM 14a – Heaving, crushing, or displacement

Reason for ruling out:

- The Rad Waste Building's foundation system is below frost level, and the interior of the building is a heated structure. The building will not be subjected to freeze/thaw cycles. Flooding did not change the frost and foundation conditions.

5.4.4.2 Detailed Assessment of Credible Potential Failure Modes

The following CPFMs are the only CPFMs carried forward for detailed assessment for the Rad Waste Building as a result of the 2011 flood. This detailed assessment is provided below.

Triggering Mechanism 7 – Soil Collapse (first time wetting)

CPFM 7a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 7b – Displaced structure/broken connections

Portions of the Rad Waste Building are supported on a differential thickness of backfill and new fill placed along the Auxiliary Building exterior wall. The thickness of a portion of this fill could be up to about 20 ft thick.

This Triggering Mechanism and CPFM could then occur as follows: the rise of the groundwater elevation associated with the flooding, in addition to the flooding that occurred when the Aqua Dam failed due to being damaged, could have resulted in the first time wetting of a portion of this backfill. When sandy soils are wetted, the water acts like a lubricant, allowing the sand particles to rearrange. When clayey soils are wetted, the water reacts with the clay, causing it to slake. When cemented soils are wetted, the water dissolves the cement, allowing the particles to rearrange.

Previous floods since backfilling of the Auxiliary Building wall have been as high as about el. 1004 ft. Rise in groundwater elevations during previous floods could have previously wetted portions of the backfill.

The following table describes observed distress indicators and other data that would increase or decrease the potential for degradation associated with this CPFM for the Rad Waste Building.

Adverse (Degradation/Direct Floodwater Impact More Likely)	Favorable (Degradation/Direct Floodwater Impact Less Likely)
Hairline and stair-step cracking of the masonry walls was observed.	Previous floods could have already wetted a majority of the fill.
	The site soils are not cemented.
	Survey data to date does not identify measurable movement.
Data Gaps: • Subsurface conditions and how they may facilitate the CPFMs are not well understood. • Additional data will be acquired from GPR, seismic survey, and geotechnical test borings.	

Conclusion

Significance

Potential for Degradation/Direct Floodwater Impact

The presence of thick fills below the Rad Waste Building may increase the potential that degradation due to these CPFMs has occurred prior to or due to the 2011 flood. Because the 2011 flood was approximately 3 ft higher than previous floods and occurred for a longer duration, the potential that the 2011 flood caused further degradation due to these CPFMs is high.

Implication

The Rad Waste Building is supported on a mat foundation that can tolerate moderate settlement. Additionally, the thickness of fill that potentially was wetted from the 2011 flood is relatively small. The occurrence of this CPFM is not expected to negatively impact the performance of the mat foundation. Therefore, the implication of the potential degradation for this CPFM is low.

Confidence

The available data are not sufficient to rule out these CPFMs or lead to a conclusion that the Rad Waste Building foundations might have been impacted because of the CPFMs. Therefore, the confidence in the above assessment is low, which means more data are necessary to draw a conclusion.

Summary

For CPFMs 7a and 7b, as discussed above, the potential for degradation is high. However, this degradation is expected to be relatively small due to previous wetting. Therefore, the combined consideration of the potential for degradation and the implications of that degradation to a structure of this type puts it in the "not significant" category. The data currently collected are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is "low," which means more data or continued monitoring and inspections might be necessary to draw a final conclusion.

5.4.5 Results and Conclusions

The CPFMs evaluated for the Rad Waste Building are presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**	CPFM 7a CPFM 7b	

5.4.6 Recommended Actions

Continued monitoring is recommended to include a continuation of elevation surveys of the previously identified targets on this structure and surrounding site. The purpose is to monitor for signs of structure distress and movement or changes in soil conditions around the structure. The results of this monitoring will be used to increase the confidence in the assessment results. Elevation surveys should be performed weekly for 4 weeks and biweekly until December 31, 2011. At the time of Revision 0, groundwater levels had not yet stabilized to nominal normal levels. Therefore, it is possible that new distress indicators could still develop. If new distress indicators are observed before December 31, 2011, appropriate HDR personnel should be notified immediately to determine whether an immediate inspection or assessment should be conducted. Observation of new distress indicators might result in a modification of the recommendations for this structure.

5.4.7 Updates Since Revision 0

Revision 0 of this Assessment Report was submitted to OPPD on October 14, 2011. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure. These assessments were incomplete in Revision 0 because the forensic investigation and/or monitoring for most of the Priority 1 Structures was not completed by the submittal date. This revision of this Assessment Report includes the results of additional forensic investigation and monitoring to date for this structure as described below.

5.4.7.1 Additional Data Available

The following additional data were available for the Rad Waste Building for Revisions 1 and 2 of this Assessment Report:

- Foundation drawings that show the Rad Waste mat foundation supported by step-tapered driven pile. These drawings are 7753-03-A-10 and 7753-03-A-17.
- Results of geophysical investigation by Geotechnology, Inc. (see Attachment 6).
- Results of geotechnical investigation by Thiele Geotech, Inc. (see Attachment 6).
- Data obtained from inclinometers by Thiele Geotech, Inc. (see Attachment 6).
- Results of continued survey by Lamp Rynearson and Associates (see Attachment 6).

5.4.7.2 Potential Failure Modes Ruled Out Prior to the Completion of the Detailed Assessment

The CPFMs ruled out in Section 5.4.4.1 were based on the understanding that the Rad Waste Building was supported by a grade-supported mat foundation. New information available for Revision 1 shows the mat foundation to be pile supported. The paragraphs below provide the ruled-out CPFMs based on the additional information available for Revisions 1 and 2 of this Assessment Report and reevaluation of each CPFM.

As previously stated, the ruled-out CPFMs reside in the Not Significant/High Confidence category and for clarity will not be shown in the Potential for Failure/Confidence matrix.

Triggering Mechanism 2 – Surface Erosion
CPFM 2b – Loss of lateral support for pile foundation

Reasons for ruling out:

- The structure was protected from the floodwater by an Aqua Dam except during a short period of time when the Aqua Dam failed due to being damaged, which allowed floodwater to enter the area inside the Aqua Dam perimeter.
- Surface erosion was not identified near the structure during the field assessments.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3b – Loss of lateral support for pile foundation (due to pumping)

Reason for ruling out:

- The structure is a sufficient distance from the known pumping locations to be outside the zone of influence of the PFM.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3e – Loss of lateral support for pile foundation (due to river drawdown)

Reason for ruling out:

- The structure is a sufficient distance from the river to be outside the zone of influence of the PFM.

Triggering Mechanism 6 – Buoyancy, Uplift Forces on Structures
CPFM 6a – Fail tension piles

Reason for ruling out:

- Distress was not observed at the structure that can be attributed to the PFM.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10d – Pile/pile group instability

Reason for ruling out:

- Machine/vibration-induced liquefaction was not observed to have occurred at the site.

Triggering Mechanism 11 – Loss of Soil Strength due to Static Liquefaction or Upward Seepage
CPFM 11d – Pile/pile group instability

Reason for ruling out:

- Machine/vibration-induced liquefaction was not observed to have occurred at the site.

5.4.7.3 Additional Analysis

The following analysis of additional data was conducted for the Rad Waste Building:

- Results of geophysical investigation report by Geotechnology, Inc.

  Seismic Refraction and Seismic ReMi tests performed around the outside perimeter of the power block identified deep anomalies that could be gravel, soft clay, loose sand, or possibly voids.

- Results of geotechnical investigation by Thiele Geotech, Inc.

  Six test borings were drilled, with continuous sampling of the soil encountered, to ground truth the Geotechnology, Inc. seismic investigation results as part of the KDI #2 forensic investigation. Test bore holes were located to penetrate the deep anomalies identified in the seismic investigation. The test boring data did not show any piping voids or very soft/very loose conditions that might be indicative of subsurface erosion/piping or related material loss or movement.

  All of the SPT and CPT test results conducted for this Assessment Report were compared to similar data from numerous other geotechnical investigations that have been conducted on the FCS site in previous years. This comparison did not identify substantial changes to the soil strength and stiffness over that time period. SPT and CPT test results were not performed in the top 10 feet to protect existing utilities.

Data from inclinometers to date, compared to the original baseline measurements, have not exceeded the accuracy range of the inclinometers. Therefore, deformation at the monitored locations since the installation of the instrumentation has not occurred.

- Results of continued survey by Lamp Rynearson and Associates.

  Survey data to date compared to the original baseline surveys have not exceeded the accuracy range of the surveying equipment. Therefore, deformation at the monitored locations, since the survey baseline was shot, has not occurred.

The CPFMs that could not be ruled out in Revision 0 are analyzed below based on the additional data available for Revisions 1 and 2 of this Assessment Report.

Triggering Mechanism 7 – Soil Collapse (first time wetting)
CPFM 7b – Displaced structure/broken connections

Portions of the Rad Waste Building are supported on a differential thickness of backfill and new fill placed along the Auxiliary Building exterior wall. The thickness of a portion of this fill could be up to about 20 ft thick. The rise of the groundwater elevation associated with the flooding, in addition to the flooding that occurred when the Aqua Dam failed due to being damaged, could have resulted in the first time wetting of a portion of this backfill. When sandy soils are wetted, the water acts like a lubricant, allowing the sand particles to rearrange. When clayey soils are wetted, the water reacts with the clay causing it to slake. When cemented soils are wetted, the water dissolves the cement allowing the particles to rearrange.

Previous floods since backfilling of the Auxiliary Building wall have been as high as about el. 1004 ft. Rise in groundwater elevations during previous floods could have previously wetted portions of the backfill.

<u>Significance</u>

Potential for Degradation/Direct Floodwater Impact

The presence of thick fills below the Rad Waste Building could increase the potential that degradation due to these CPFMs has occurred prior to or due to the 2011 flood. Because the 2011 flood was approximately 3 ft higher than previous floods and occurred for a longer duration, the potential that the 2011 flood caused further degradation due to the CPFM is high.

Implication

The structures supported on the grade would settle while structures supported by piles would not. This could result in a "pinch-point at the interface of the non-pile supported and pile supported structures. Depending on the flexibility of the interface connection and the magnitude of settlement, the occurrence of the CPFM could impact the performance of the structure negatively. The amount of settlement due to collapse of the upper 3 ft of fill is negligible. Therefore, the implication of the potential degradation for the CPFM is low.

Confidence

The available data lead to a conclusion that the Rad Waste Building was not impacted by the CPFM. Therefore, the confidence in the above assessment is high.

Summary

For CPFM 7b as discussed above, the potential for degradation is high. However, this degradation is expected to be relatively small due to previous wetting. Therefore, the combined consideration of the potential for degradation and the implications of that degradation to a structure of this type put it in the "not significant" category. The data currently collected are sufficient to rule out these CPFMs. Therefore, the confidence in the above assessment is "high."

Triggering Mechanism 7 – Soil Collapse (first time wetting)
CPFM 7d – Piles buckling from down drag

Portions of the Rad Waste Building are supported on a differential thickness of backfill and new fill placed along the Auxiliary Building exterior wall. The thickness of a portion of this fill could be up to about 20 ft thick. The rise of the groundwater elevation associated with the flooding, in addition to the flooding that occurred when the Aqua Dam failed due to being damaged, could have resulted in the first time wetting of a portion of this backfill. When sandy soils are wetted, the water acts like a lubricant, allowing the sand particles to rearrange. When clayey soils are wetted, the water reacts with the clay causing it to slake. When cemented soils are wetted, the water dissolves the cement allowing the particles to rearrange.

Previous floods since backfilling of the Auxiliary Building wall have been as high as about el. 1004 ft. Rise in groundwater elevations during previous floods could have previously wetted portions of the backfill.

Significance

Potential for Degradation/Direct Floodwater Impact

The presence of thick fills below the Rad Waste Building could increase the potential that degradation due to the CPFM has occurred prior to or due to the 2011 flood. Because the 2011 flood was approximately 3 ft higher than previous floods and occurred for a longer duration, the potential that the 2011 flood caused further degradation due to the CPFM is high.

Implication

The amount of down drag force that would be applied to the piles due to collapse of the upper 3 ft of fill is negligible. The occurrence of the CPFM is not expected to impact the performance of the mat foundation negatively. Therefore, the implication of the potential degradation for the CPFM is low.

Confidence

The available data lead to a conclusion that the Rad Waste Building foundation was not impacted by the CPFM. Therefore, the confidence in the above assessment is high.

Summary

For CPFM 7d, as discussed above, the potential for degradation is high. However, this degradation is expected to be relatively small due to previous wetting. Therefore, the combined consideration of the potential for degradation and the implications of that degradation to a structure of this type put it in the "not significant" category. The data currently collected are sufficient to rule out the CPFM. Therefore, the confidence in the above assessment is "high."

5.4.7.1 Revised Results

The CPFM evaluated for the Rad Waste Building is presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation. CPFMs 7b and 7d for the Rad Waste Building are not associated with Key Distress Indicators. The results of the additional forensic investigation show that these CPFMs are ruled out. Therefore, assuming that no further concerns are identified through the monitoring program for the Rad Waste Building (discussed in Section 5.4.6 and continuing until December 31, 2011), these CPFMs are placed in the quadrant of the matrix representing "No Further Action Recommended Related to the 2011 Flood."

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**		CPFM 7b CPFM 7d

5.4.7.2 Conclusions

In the assessment of the FCS Structures, the first step was to develop a list of all Triggering Mechanisms and PFMs that could have occurred due to the prolonged inundation of the FCS site during the 2011 Missouri River flood and could have negatively impacted these structures. The next step was to use data from various investigations, including systematic observation of the structures over time, either to eliminate the Triggering Mechanisms and PFMs from the list or to recommend further investigation and/or physical modifications to remove them from the list for any particular structure. Because all CPFMs for the Rad Waste Building other than CPFMs 7b and 7d had been ruled out prior to Revision 1, and because CPFMs 7b and 7d have been ruled out as a result of the Revision 1 findings, no Triggering Mechanisms and their associated PFMs remain credible for the Rad Waste Building. Therefore, HDR has concluded that the 2011 Missouri River flood did not impact the geotechnical and structural integrity of the Rad Waste Building because the potential for failure of this structure due to the flood is not significant.

Section 5.5

Technical Support Center

5.5 Technical Support Center

5.5.1 Summary of Technical Support Center

Baseline information for the Technical Support Center is provided in Section 2.0, Site History, Description, and Baseline Condition.

5.5.2 Inputs/References Supporting the Analysis

Table 5.5-1 lists references provided by OPPD and other documents used to support HDR's analysis.

Table 5.5-1 - References for Technical Support Center

Document Title	OPPD Document Number (if applicable)	Date	Page Number(s)
Mat-Plan Sections and Details	4778-293-404-001 (#31553)	2/16/1984	
Foundation Walls el. 1005 ft	4778-293-405-002 (#31554)	2/16/1984	
Sections and Details	4778-293-108-001 (#31547)	6/20/1980	
Naval Facilities Engineering Command, Design Manual 7.01, Soil Mechanics		9/1986	All

Detailed site observations—field reports, field notes, and inspection checklists—for the Technical Support Center are provided in Attachment 8.

Observed performance and pertinent background data are as follows:

- This structure is surrounded on all sides by the Auxiliary Building, the CARP, and the Maintenance Shop. The CARP and Maintenance Shop have shallow foundations similar to the Phase 2 expansion of the Technical Support Center. The Auxiliary Building, located to the south, is supported by a deep foundation system with a basement.
- The structure is designated as a Class I (seismic) structure in accordance with OPPD.
- The foundation for Phase 1 consists of a 2.0-ft minimum thickness rigid structural mat slab with top-of-concrete elevation of 1004.0 ft (see drawing 4778-293-404-001). Finished floor elevation is 1005.0 ft, which is achieved through the use of an architectural false floor or concrete fill, depending on the specific location.
- The foundation for Phase 2 consists of simple wall footings and stem walls and a slab-on-grade with top-of-concrete elevation of 1005.0 ft (see drawing 4778-293-405-002).
- The superstructure for Phase 1 consists of cast-in-place concrete walls and roof slab. The roof system is a membrane roof with tapered insulation (see drawing 4778-293-108-001).
- The superstructure for Phase 2 consists of concrete masonry walls. The roof is an open-webbed joist system with concrete slab on metal deck. The roof system is the same as for Phase 1 (see drawing 4778-293-108-001).
- The drawings indicate a 1-in.-wide expansion joint at the floor and roof elevations (see drawing 4778-293-108-001).
- This structure, along with the surrounding buildings, was protected from the 2011 flood by an Aqua Dam. It is possible that the foundations for this structure were subjected to high groundwater

pressure equal to the flood elevation. The maximum flood elevation at the Aqua Dam near the Technical Support Center was approximately el. 1007 ft. No incident report summaries or inspection records are available for this structure.

- No design basis summary document is available for this structure.
- General observations of the interior of the structure were limited by the accessibility of certain rooms. In addition, many areas have architectural walls and ceilings that limit visual observations.
- Where the concrete slab was accessible, there were no signs of cracking, movement, or water infiltration at the time of this inspection.
- Indications of structural distress in many areas were limited to the observation of indicators within the architectural treatments such as gypsum board walls and ceilings. No indicators were found within the architectural systems.
- Sandbags were stacked within the corridor outside the mechanical chase and locker rooms. There was no sign of water infiltration through slab joints at the time of inspection.
- Voids were found below the slab in the Turbine Building, which is located to the southeast of the Technical Support Center. For further information, see Section 5.8. A more detailed discussion of this Key Distress Indicator is presented in Section 4.1.
- The Maintenance Shop to the northeast has documented settlement issues. One building column footing and a section of floor slab had settled at the time of Revision 0. A more detailed discussion of this Key Distress Indicator is presented in Section 4.3.

5.5.3 Assessment Methods and Procedures

5.5.3.1 Assessment Procedures Accomplished

Assessments of the Technical Support Center included the following:

- Visual inspection of accessible areas of the interior of the structure
- Review of previously referenced documents listed in Table 5.5-1

5.5.3.2 Assessment Procedures Not Completed

Assessments of the Technical Support Center that were not completed included the following:

- Baseline survey with periodic review indicating trends in the top of concrete. This was not completed because the structure is surrounded by other structures and was not directly accessible to survey.
- Geotechnical borings in the vicinity of the Technical Support Center to determine current soil conditions and capacities.

5.5.4 Analysis

Identified PFMs were initially reviewed as discussed in Section 3.0. The review considered the preliminary information available from OPPD data files and from initial walk-down observations. Eleven PFMs associated with five different Triggering Mechanisms were determined to be "non-credible" for all Priority 1 Structures, as discussed in Section 3.6. The remaining PFMs were carried forward as "credible." After the design review for each structure, the structure observations, and the results of available geotechnical, geophysical, and survey data were analyzed, a number of CPFMs were ruled out as discussed in Section 5.5.4.1. The CPFMs carried forward for detailed assessment are discussed in Section 5.5.4.2.

5.5.4.1 Potential Failure Modes Ruled Out Prior to the Completion of the Detailed Assessment

The ruled-out CPFMs reside in the Not Significant/High Confidence category and for clarity will not be shown in the Potential for Failure/Confidence matrix.

Triggering Mechanism 2 – Surface Erosion

CPFM 2a – Undermining shallow foundation/slab/surfaces

Reason for ruling out:

- The Technical Support Center is completely surrounded by other structures and is therefore not subjected to surface erosion.

Triggering Mechanism 3 – Subsurface Erosion/Piping

CPFM 3d – Undermining and settlement of shallow foundation/slab (due to river

Reason for ruling out:

- The Technical Support Center is not near the riverbank and is surrounded by other structures and is therefore not subjected to undermining due to river drawdown.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction

CPFM 10a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 10b – Displaced structure/broken connections
CPFM 10c – Additional lateral force on below-grade walls

Reason for ruling out:

- The Technical Support Center does not have permanent equipment capable of providing enough energy to result in vibration-induced liquefaction.

Triggering Mechanism 11 – Loss of Soil Strength due to Static Liquefaction or Upward Seepage

CPFM 11a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 11b – Displaced structure/broken connections
CPFM 11c – Additional lateral force on below-grade walls

Reason for ruling out:

- Static liquefaction was not observed on site in the vicinity of the Technical Support Center and surrounding structures.

Triggering Mechanism 12 – Rapid Drawdown

CPFM 12a – River bank slope failure and undermining surrounding structures
CPFM 12b – Lateral spreading

Reason for ruling out:

- The Technical Support Center is located a sufficient distance away from the river bank and therefore is outside the zone of influence of a bank slope failure or lateral spreading.

Triggering Mechanism 14 – Frost Effects
CPFM 14a – Heaving, crushing, or displacement

Reasons for ruling out:

- The Technical Support Center foundation system is below frost level, and the interior of the building is a heated structure. The building will not be subjected to freeze/thaw cycles. Therefore, frost effects have been discounted.

5.5.4.2 Detailed Assessment of Credible Potential Failure Modes

The following CPFMs are the only CPFMs carried forward for detailed assessment for the Technical Support Center as a result of the 2011 flood. This detailed assessment is provided below.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3a – Undermining and settlement of shallow foundation/slab/surfaces (due to pumping)

The Turbine Building, which is located to the southeast of the Technical Support Center, has documented history of a void below the foundation slab dating back to 1997. This void was confirmed via cored holes in the foundation slabs and camera recordings of broken drain piping under the floor slab. Conversations with OPPD personnel indicate that groundwater has been flowing at varying rates through these broken pipes into the sump from that time to the present day. The rate of flow into the sump is directly related to the hydraulic head of the groundwater. As the floodwater increased in elevation across the site, observed flow rates increased. The flow of groundwater into this drain piping system through the breaks in the pipes is one of the Key Distress Indicators discussed in Section 4. This drain pipe system was designed as a closed system; therefore, the pipes are not surrounded by appropriate filter systems to preclude the transportation of soils from the surrounding area under the slab. It is logical to assume that because the groundwater moves below the foundation and into the broken piping, some movement of the soil has occurred. If these voids were to continue under the Technical Support Center they could be large enough to undermine the shallow foundations or slab on grade.

The Triggering Mechanism and CPFM could then occur as follows: the unfiltered seepage condition will remain until the breaks in the piping system are repaired, which means the potential for further erosion remains. Erosion could extend out, creating large voids under the Turbine Building mat foundation and ultimately under the Technical Support Center Foundation.

The following table describes observed distress indicators and other data that would increase or decrease the potential for degradation associated with this CPFM for the Technical Support Center.

Adverse (Degradation/Direct Floodwater Impact More Likely)	Favorable (Degradation/Direct Floodwater Impact Less Likely)

A documented void exists under the foundation slab of the Turbine Building with a known hydraulic connection between groundwater elevation and flows into the building sump. A more detailed discussion of this Key Distress Indicator is presented in Section 4.1.	There have been no observed signs of structural distress in the floor slab or indicators of structural distress in the architectural coverings at the current loading conditions.
Unknown soil compaction density below the structure.	
Varying foundation systems within the same structure (mat vs. spread footing) have the potential for differential settlement.	
The Maintenance Shop, to the east, has documented settlement issues. One building column footing and a section of floor slab had settled at the time of Revision 0. A more detailed discussion of this Key Distress Indicator is presented in Section 4.3.	
Data Gaps: • Previous areas that were not accessible will be inspected. • Continued observation of structure as the flood waters recede will be performed.	

Conclusion

<u>Significance</u>

Potential for Degradation/Direct Floodwater Impact

Indicators for this CPFM have been observed in the Turbine Building, which is located to the southeast of the Technical Support Center. Voids below the base slab in the Turbine Building are known to exist and heavy flows of groundwater are being pumped from the sump. Because the 2011 flood caused increased flow through the broken drain pipes, the potential that the flood caused further and more rapid degradation due to this CPFM is high. It is possible that these voids extend under the Technical Support Center.

Implication

The occurrence of this CPFM could cause settlement of the shallow foundations or slab on grade and has the potential to cause cracking of the walls, cracking of the slabs, or distress to the architectural coverings. However, the Phase 1 portion, which is designated Class I, is founded on a mat foundation and has much more redundancy. If degradation occurred it would be slower to develop and would allow time to respond with corrective action. Minor amounts of settlement would be considered a serviceability problem, not a strength or safety issue. Therefore, this implication of the potential degradation for this CPFM is low.

<u>Confidence</u>

The available data are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is low, which means more data are necessary to draw a conclusion.

<u>Summary</u>

For CPFM 3a, as discussed above, the combined consideration of the potential for degradation and the implications of that degradation to a structure of this type puts it in the "not significant" category. It is possible that voids extend under the Technical Support Center, although the potential is low due to the distance from the Technical Support Center to the sump in the Turbine Building. The data currently collected are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is low, which means more data or continued monitoring and inspections might be necessary to draw a conclusion.

Triggering Mechanism 6 – Buoyancy, Uplift Forces on Structures

CPFM 6b – Cracked slab, loss of structural support
CPFM 6c – Displaced structure/broken connections

The peak flood elevation prior to 2011 was 1003.3 ft, which occurred in 1993. The peak flood elevation in 2011 was approximately 1006.9 ft.

The Triggering Mechanism and CPFMs could occur as follows: water level rises in areas around the Technical Support Center. Water is pumped from inside the protected area, causing an uplift force. This uplift force is reduced by the head loss between the flooded areas and the area under the building. This uplift force exceeds the self weight of the structure, causing structure slabs to crack and buckle. Additional damage could include structure displacement and broken interior utility connections.

The following table describes observed distress indicators and other data that would increase or decrease the potential for degradation associated with these CPFMs for the Technical Support Center.

Adverse (Degradation/Direct Floodwater Impact More Likely)	Favorable (Degradation/Direct Floodwater Impact Less Likely)
Sandbags in the corridor adjacent to the restrooms indicate that water intrusion into the structure was happening at some time during the 2011 flood. This water could be coming up through slab joints or backing up through the floor drain system.	There have been no observed signs of structural distress in the floor slab or indicators of structural distress in the architectural coverings at the current loading conditions.
	Floodwater levels are receding. The structure has already experienced the maximum buoyant uplift pressures. Therefore, the possibility of failure from buoyancy is reduced.
Data Gaps: • Previous areas that were not accessible due to security issues will be inspected. • Visual observation of structural elements that were not accessible. • Continued observation of structure as the flood waters recede will be performed.	

Conclusion

Significance

Potential for Degradation/Direct Floodwater Impact

The degradation associated with these CPFMs would include vertical movement of subgrade soils through the slab joints within the structure. There have been no observed signs of structural distress in the floor slab or indicators of structural distress in the architectural coverings at the current loading conditions. Sandbags in the corridor adjacent to the restrooms could indicate that water intrusion into the structure was happening at some time during the 2011 flood. This water could be coming up through slab joints or backing up through the floor drain system. Since there were not signs of structure distress, the potential for degradation is low.

Implication

The occurrence of these CPFMs could cause vertical movement of the foundation or slab on grade and has the potential to cause distress to the structure such as cracking of the walls, cracking of the slabs, or distress to the architectural coverings. In addition, positive upward pressure can cause water infiltration into the structure. The degradation is considered a serviceability problem, not a strength or safety issue. Therefore the implication of the potential degradation for these CPFMs is low.

Confidence

Indicators for this CPFM have not been observed; however, visual observation was limited to a few accessible rooms and the main corridor. The available data are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is low, which means more data are necessary to draw a conclusion.

Summary

For CPFMs 6b and 6c, as discussed above, the combined consideration of the potential for degradation and the implications of that degradation to a structure of this type puts it in the "not significant" category. The structure has already experienced the maximum buoyant uplift pressures, and there have been no observed signs of structural distress in the floor slab or indicators of structural distress in the architectural coverings at these loading conditions. The data currently collected are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is low, which means more data or continued monitoring and inspections might be necessary to draw a conclusion.

Triggering Mechanism 7 – Soil Collapse (first time wetting)

CPFM 7a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 7b – Displaced structure/broken connections
CPFM 7c – General site settlement

The Triggering Mechanism and CPFMs could occur as follows: soil material under the structure is poorly compacted backfill or uncompacted native subgrade. Groundwater elevation rises to a level that saturates these soils. Soil undergoes excessive settlement, termed "collapse settlement," due to first time wetting. The potential damage includes settlement of floor slabs and foundations, cracks in walls, and deflections in floors and roofs.

The peak flood elevation prior to 2011 was 1003.3 ft, which occurred in 1993. The peak flood elevation in 2011 was approximately 1006.9 ft. The bottom of foundation elevation for the Phase 1 rigid mat is 1000 to 1002, which is below the previously documented flood elevation. The bottom of the Phase 2 slab on grade is approximately 1004.33 ft, which is above the previously documented high-water level but within the flood elevation of the current year. Therefore, there is approximately 1 ft of sub-grade directly below the slab on grade that has the potential to be subjected to first time wetting.

The following table describes observed distress indicators and other data that would increase or decrease the potential for degradation associated with these CPFMs for the Technical Support Center.

Adverse (Degradation/Direct Floodwater Impact More Likely)	Favorable (Degradation/Direct Floodwater Impact Less Likely)
Unknown backfill compaction density below the structure.	There have been no observed signs of structural distress in the floor slab or indicators of structural distress in the architectural coverings at the current loading conditions.
Sandbags in the corridor indicate that water has potentially moved through the floor drain piping, through piping trenches, or upward through slab joints.	The peak flood elevation prior to 2011 was 1003.3 ft, which occurred in 1993. The peak flood elevation in 2011 was approximately 1006.9 ft. Soils below this structure were potentially wetted during earlier flooding events.
The Maintenance Shop to the northeast has documented settlement issues. One building column footing and a section of floor slab had settled at the time of Revision 0. A more detailed discussion of this Key Distress Indicator is presented in Section 4.3.	
Data Gaps: • Survey data to track trends in vertical movement of the structure. • Visual observation of structural elements that were not accessible. • Previous areas that were not accessible will be inspected. • Continued observation of structure as the flood waters recede will be performed.	

Conclusion

Significance

Potential for Degradation/Direct Floodwater Impact

Indicators for these CPFMs have not been observed in the Technical Support Center. However, survey data has not been obtained to verify that vertical movement has not occurred. The peak flood elevation prior to 2011 was 1003.3 ft, which occurred in 1993. The peak flood elevation in 2011 was approximately 1006.9 ft. Soils below this structure were potentially wetted during earlier flooding events. The potential for degradation is considered to be low.

Implication

The occurrence of this CPFM could cause settlement of the shallow foundations or slab on grade and has the potential to cause issues with the structure such as cracking of the walls, cracking of the slabs, or distress to the architectural coverings. However, the Phase 1 portion, which is designated Class I, is founded on a mat foundation and has much more redundancy. If degradation occurred it would be slower to develop and would allow time to respond with corrective action. Minor amounts of settlement would be considered a serviceability problem, not a strength or safety issue. The layers of subgrade not previously wetted are likely thin, reducing the effects of first time wetting. Therefore, the implication of the potential degradation for these CPFMs is low.

Confidence

Indicators for this CPFM have not been observed; however, visual observation was limited to a few accessible rooms and the main corridor. The available data are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is low, which means more data are necessary to draw a conclusion.

Summary

For CPFMs 7a through 7c, as discussed above, the combined consideration of the potential for degradation and the implications of that degradation to a structure of this type puts it in the "not significant" category. The data currently collected are not sufficient to rule out these CPFMs. Therefore, the confidence in the above assessment is low, which means more data or continued monitoring and inspections might be necessary to draw a conclusion.

5.5.5 Results and Conclusions

The CPFMs evaluated for the Technical Support Center are presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**	CPFM 3a CPFM 6b CPFM 6c CPFM 7a CPFM 7b CPFM 7c	

5.5.6 Recommended Actions

The following actions are recommended for the Technical Support Center:

- Visual inspection of the previously non accessible areas.
- Further forensic investigations and physical modifications are recommended to address CPFM 3a, 7a though 7c (Key Distress Indicator #1 and #3). These recommendations are described in detail in Section 4.1.3.

Continued monitoring is recommended to consist of visual inspection of the Technical Support Center. The purpose is to monitor for signs of structure distress and movement or changes in soil conditions around the structure. The results of this monitoring will be used to increase the confidence in the assessment results. The visual inspections should be performed weekly for 4 weeks and biweekly until December 31, 2011.

At the time of Revision 0, groundwater levels had not yet stabilized to nominal normal levels. Therefore, it is possible that new distress indicators could still develop. If new distress indicators are observed before December 31, 2011, appropriate HDR personnel should be notified immediately to determine if an immediate inspection or assessment should be conducted. Observation of new distress indicators might result in a modification of the recommendations for this structure.

5.5.7 Updates Since Revision 0

Revision 0 of this Assessment Report was submitted to OPPD on October 14, 2011. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure. These assessments were incomplete in Revision 0 because the forensic investigation and/or monitoring for most of the Priority 1 Structures was not completed by the submittal date. This revision of this Assessment Report includes the results of additional forensic investigation and monitoring to date for this structure as described below.

5.5.7.1 Additional Data Available

The following additional data were available for the Technical Support Center for Revisions 1 and 2 of this Assessment Report:

- Results of KDI #1 forensic investigation (see Section 4.1)
- Results of KDI #3 forensic investigation (see Section 4.3)
- Additional groundwater monitoring well and river stage level data from OPPD.
- Results of geotechnical investigation by Thiele Geotech, Inc. (see Attachment 6).
- Results of continued survey by Lamp Rynearson and Associates (see Attachment 6).
- Results of continued monitoring and assessments in areas previously observed
- Field assessments of the areas of the Technical Support Center not previously observed.
- Results of crack monitor observations in Room 127

5.5.7.2 Additional Analysis

The following analysis of additional data was conducted for the Technical Support Center:

- Groundwater monitoring well and river stage level data from OPPD.

  Data shows that the river and groundwater have returned to nominal normal levels.

- Results of geotechnical investigation by Thiele Geotech, Inc.

  All of the SPT and CPT test results conducted for this Assessment Report were compared to similar data from numerous other geotechnical investigations that have been conducted on the FCS site in previous years. This comparison did not identify substantial changes to the soil strength and stiffness over that time period. SPT and CPT test results were not performed in the top 10 feet to protect existing utilities.

- Results of continued survey by Lamp Rynearson and Associates.

  Survey data to date compared to the original baseline surveys have not exceeded the accuracy range of the surveying equipment. Therefore, deformation at the monitored locations, since the survey baseline was shot, has not occurred.

Several CPFMs were identified in Revision 0. Since Revision 0, additional data have become available that have clarified the significance and confidence for these CPFMs. The following presents each of the previously identified CPFMs and the new interpretation of their significance and confidence based on the new data.

Field observations of the Technical Support rooms (not previously visited due to security issues) identified a horizontal crack along the east wall of Room 127. This CMU wall is along the east expansion joint line with the adjacent Maintenance Shop. The horizontal crack in the east wall was within a horizontal mortar joint and was approximately 15 ft long and up to about 1/8 in. wide. The crack could be caused by localized settlement of the foundation below the wall or flexural cracking of the wall due to out-of-plane forces. The crack appears to be new since there is paint that bridges across the crack and there does not appear to be an accumulation of dust within the opening. There is no evidence of out-of-plane movement of the wall or other signs of structural distress at this location.

Field observations of the Technical Support rooms (not previously visited due to security issues) identified cracking along the east wall of the crawlspace just south of Room 127. This CMU wall is along the east expansion joint line with the adjacent Maintenance Shop and the south expansion joint line with the Auxiliary Building. This expansion joint is separating a basement with pile foundation for the Auxiliary Building from shallow footings and slab on grade for the Technical Support Center. As the shallow foundations settled independent of the adjacent structures the settlement caused a point load on the CMU wall causing vertical cracking.

These detailed observations indicate that some foundation and slab on grade movement has occurred along the expansion joints between the Technical Support Center and the adjacent buildings. However, no out-of-plane movement of the CMU walls or other signs of structural distress have been observed at this location.

Triggering Mechanism 3 – Subsurface Erosion/Piping

CPFM 3a – Undermining and settlement of shallow foundation/slab/surfaces (due to pumping)

Significance

Potential for Degradation/Direct Floodwater Impact

Indicators for this CPFM have been observed in the Turbine Building, Maintenance Shop, and the masonry wall of the Tech Support Center shared with the Maintenance Shop. Voids below the slab in the Turbine Building and the Maintenance Shop are known to exist and might extend below the foundations for the masonry walls causing it to settle. The crack in the masonry wall and settlement is thought to be related to KDIs 1 and 3. The potential for degradation to occur at this time is high.

Implication

The occurrence of this CPFM could cause settlement of the shallow foundations or slab on grade and has the potential to cause cracking of the walls, cracking of the slabs, or distress to the architectural coverings. However, the Phase 1 portion, which is designated Class I, is founded on a mat foundation and has much more redundancy. If degradation occurred it would be slower to develop and would allow time to respond with corrective action. Minor amounts of settlement would be considered a serviceability problem, not a strength or safety issue. Therefore, this implication of the potential degradation for this CPFM is low.

Confidence

The occurrence of damage due to subsurface erosion was not known at the time of Revision 0 due to lack of access to some of the structure. Subsequent field inspections indicate structure movement that is thought to be associated with this CPFM and directly related to KDIs 1and 3. If repairs are conducted relating to KDIs 1 and 3 then the confidence of the assessment for this CPFM becomes high.

Summary

For CPFM 3a, as discussed above, the future potential for degradation is high because signs of distress were observed. The combined consideration of the future potential for degradation and the implications of that degradation to the structure put it in the "significant" category. The data collected since Revision 0 are sufficient to rule out this CPFM assuming the repairs under KDIs 1 and 3 are conducted. Therefore, the confidence in the above assessment is high, which means no additional data and inspections are necessary to draw a conclusion.

Triggering Mechanism 6 – Buoyancy, Uplift Forces on Structures

CPFM 6b – Cracked slab, loss of structural support
CPFM 6c – Displaced structure/broken connections

Significance

Potential for Degradation/Direct Floodwater Impact

There have been no observed signs of structural distress in the floor slab or indicators of structural distress in the architectural coverings at the current loading conditions that would be accredited to these CPFMs. Since there were no signs of structure distress in these areas, the potential for degradation is low.

Implication

The occurrence of these CPFMs could cause vertical movement of the foundation or slab on grade and has the potential to cause distress to the structure such as cracking of the walls, cracking of the slabs, or distress to the architectural coverings. In addition, positive upward pressure can cause water infiltration into the structure. The degradation is considered a serviceability issue, not a life safety issue. Therefore, the implication of the potential degradation for these CPFMs is low.

Confidence

Since Revision 0 was completed, areas that were not previously accessible in the Tech Support Center have been observed. Although some signs of distress have been observed in these areas, it is not believed that it could be related to these CPFMs because the wall footings and slab on grade did not show signs of distress related to buoyancy or uplift forces. Since all areas of the Tech Support Center have now been observed and no signs of distress relating to these CPFMs have been found, the confidence of the assessment for these CPFMs is high.

Summary

For CPFMs 6b and 6c, as discussed above, the potential for degradation is low because the implications of these types of CPFMs would most likely be serviceability issues rather than life safety issues and would be apparent at this time. The combined consideration of the potential for degradation and the implications of that degradation to a structure of this type puts it in the "not significant" category. The data collected since Revision 0 are sufficient to rule out this CPFM. Therefore, the confidence in the assessment is high, which means no additional data and inspections are necessary to draw a conclusion.

Triggering Mechanism 7 – Soil Collapse (first time wetting)

CPFM 7a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 7b – Displaced structure/broken connections
CPFM 7c – General site settlement

Significance

Potential for Degradation/Direct Floodwater Impact

Indicators for these CPFMs may have been observed in the Technical Support Center in Room 127 where the horizontal masonry crack exists. CPFMs 7a through 7c as they pertain to the Technical Support Center are further addressed under KDI #3 in Section 4.3 and were determined to not be the likely cause of the observed distress. Therefore, the potential for degradation to occur at this time is low.

Implication

The occurrence of these CPFMs would cause settlement of the shallow foundations or slab on grade and has the potential to cause issues with the structure such as cracking of the walls, cracking of the slabs, or distress to the architectural coverings. However, the Phase 1 portion, which is designated Class I, is founded on a mat foundation and has much more redundancy. If degradation occurred it would be slower to develop and would allow time to respond with corrective action. Minor amounts of settlement would be considered a serviceability problem, not a strength or safety issue. The layers of subgrade not previously wetted are likely thin, reducing the effects of first time wetting. Therefore, the implication of the potential degradation for these CPFMs is low.

Confidence

The occurrence of damage due to soil collapse was not known at the time of Revision 0 due to lack of access to some of the structure. Subsequent field inspections indicate structure movement that can be associated with these CPFMs. The investigation of KDI #3 identified the distress as being the result of subsurface erosion due groundwater flowing into the broken drain pipes below the Turbine Building floor. Therefore, the distress is not believed to be the result of soil collapse due to first time wetting. The confidence of the assessment for these CPFMs becomes high.

Summary

Since Revision 1, KDI #3 was investigated and concluded that the distress in the Technical Support Center is most likely the result of subsurface erosion due groundwater flowing into the broken drain pipes below the Turbine Building floor.

For CPFMs 7a through 7c, as discussed above, our confidence is high that the future potential for degradation is low because soil collapse due to first time wetting is not believed to have caused the distress. The data collected since Revision 0 are sufficient to rule out these, which means no additional data and inspections are necessary to draw a conclusion.

5.5.7.1 Revised Results

The CPFMs evaluated for the Technical Support Center are presented in the following matrix, which shows the rating for estimated significance and the level of confidence in the evaluation. CPFMs 6b and 6c for the Technical Support Center are not associated with Key Distress Indicators. The results of the additional monitoring show that these CPFMs are ruled out. The results of the KDI #3 investigations show that CPFMs 7a, 7b, and 7c are not associated with Key Distress Indicators and can be ruled out. Therefore, assuming that no further concerns are identified through the monitoring program for the Technical Support Center (discussed in Section 5.5.6 and continuing until December 31, 2011), these CPFMs are moved to the quadrant of the matrix representing "No Further Action Recommended Related to the 2011 Flood." CPFM 3a is associated with Key Distress Indicators #1 and #3. Sections 4.1 and 4.3 present the results of additional forensic investigation that was conducted to ascertain whether these CPFMs could be ruled out. The results of the additional forensic investigations show that if the recommendations for physical modifications in KDI #1 and KDI #3 are implemented that this CPFM is ruled out. Therefore, assuming that no further concerns are identified through the monitoring program for the Technical Support Center (discussed in Section 5.5.6 and continuing until December 31, 2011), these CPFMs are moved to the quadrant of the matrix representing "No Further Action Recommended Related to the 2011 Flood."

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**		CPFM 3a CPFM 6b CPFM 6c CPFM 7a CPFM 7b CPFM 7c

5.5.7.2 Conclusions

In the assessment of the FCS Structures, the first step was to develop a list of all Triggering Mechanisms and PFMs that could have occurred due to the prolonged inundation of the FCS site during the 2011 Missouri River flood and could have negatively impacted these structures. The next step was to use data from various investigations, including systematic observation of the structures over time, either to eliminate the Triggering Mechanisms and PFMs from the list or to recommend further investigation and/or physical modifications to remove them from the list for any particular structure. Because all CPFMs for the Technical Support Center other than CPFMs 3a, 6b, 6c, 7a, 7b, and 7c had been ruled out prior to Revision 1, and because CPFMs 6b, 6c, 7a, 7b and 7c had been ruled out as a result of the Revision 1 findings, and because CPFM 3a will be ruled out when the physical modifications recommended for KDIs #1 and #3 in Section 4.1 and 4.3 are implemented, no Triggering Mechanisms and their associated PFMs remain credible for the Technical Support Center. HDR has concluded that the geotechnical and structural impacts of the 2011 Missouri River flood will be mitigated by the implementation of the physical modifications recommended in this Assessment Report. Therefore, after the implementation of the recommended physical modifications, the potential for failure of this structure due to the flood will not be significant.

Section 5.6

Independent Spent Fuel Storage Installation (ISFSI)

Omaha Public Power District

Draft

5.6 Independent Spent Fuel Storage Installation

5.6.1 Summary of Independent Spent Fuel Storage Installation

Baseline information for the Independent Spent Fuel Storage Installation (ISFSI) is provided in Section 2.0, Site History, Description, and Baseline Condition.

The ISFSI consists of spent fuel modules placed inside 3-ft-thick reinforced concrete shield walls and ceiling. The modules and shield walls are supported on a 2-ft-thick reinforced concrete basemat. Approach slabs are located on the plan north, south, and east sides of the basemat. The approach slabs consist of approximately 0.7-ft-thick reinforced concrete. A haul road exits at the plan northeast corner of the approach slabs and turns ninety degrees to exit toward the west. At the end of the radius, the concrete pavement ends and gravel surfacing begins.

The basemat is elevated relative to the surrounding grades to provide protection from flooding. The elevation at the surface of the basemat is about 1009.5 ft. The approach slab slopes downward away from the basemat to provide drainage. The haul road slopes downward to the surrounding grade, which is at about el. 1004 ft. Side slopes along the perimeter of the elevated area are protected from erosion with large-diameter riprap. The riprap extends from the edge of the pavement down to the toe of the slope.

An Electrical Equipment Building is located southeast of the ISFSI. A cable trench extends from the Electrical Equipment Building to the existing New Warehouse and from the Electrical Equipment Building to the spent fuel modules. The trench follows a path from the Electrical Equipment Building plan west along the plan south edge of the approach slab and then turns plan north along the plan west edge of the approach slab, where it enters the shield walls.

Two high mast light towers are located near the ISFSI. One is near the toe of the side slope along the plan south side of the ISFSI, and one is near the toe of the slope between the ISFSI and the haul road.

The basemat and approach slabs are grade supported. Site preparation prior to placement of the basemat and approach slabs included over-excavation of the existing fill. The structural backfill and structural fill consisted of crushed limestone compacted to 95 percent of the material's maximum density as determined by the modified Proctor test (ASTM D 1557) at a water content between 3 percent below and 3 percent above optimum water content.

The ISFSI—including the haul road ramp, the Electrical Equipment Building, and two high mast light towers—is surrounded by an independent security fence.

5.6.2 Inputs/References Supporting the Analysis

Table 5.6-1 lists references provided by OPPD and other documents used to support HDR's analysis.

Table 5.6-1 - References for ISFSI

Document Title	OPPD Document Number (if applicable)	Date	Page Number(s)
Geotechnical Report Independent Spent Fuel Storage Installation Fort Calhoun Station	58209-G(D)-3, Rev. 0	4/28/2004	All
Fort Calhoun Station ISFSI, Basemat Evaluation	59058-E(D)-1, Rev. 0	7/7/2004	All
Naval Facilities Engineering Command, Design Manual 7.01, Soil Mechanics		9/1986	All

Detailed site observations—field reports, field notes, and inspection checklists—for the ISFSI are provided in Attachment 8.

Observed performance and pertinent background data are as follows:

- Floodwaters extended about half-way up the side slopes of the ISFSI platform.
- The Electrical Equipment Building was protected by a temporary berm constructed with sandbags, and a pump appeared to have been used for a period.
- Water stains on the Electrical Equipment Building show evidence of the structure being inundated by floodwater.
- The river bank is armored and has historically protected and stabilized the existing river bank.
- USACE reduced Missouri River Mainstem System releases to 40,000 cfs on October 2, 2011. River levels corresponding to the 40,000 cfs release rate stabilized at FCS on October 4, 2011, at about el. 995 feet.

5.6.3 Assessment Methods and Procedures

5.6.3.1 Assessment Procedures Accomplished

Assessments of the ISFSI included the following:

- A visual inspection of the grade-supported slabs and surrounding grades
- Probing of the grades around the perimeter of the structure for changes in consistency
- An assessment of collected survey data to date for indications of trends in the movement of the structure
- A review of building plans and the geotechnical report to identify possible subsurface features that might be susceptible to the PFMs

5.6.3.2 Assessment Procedures Not Completed

Assessments of the ISFSI that were not completed include the following:

- Geophysical surveys using GPR and seismic refraction to find voids (currently not planned. Other data and observations are sufficient to reach a confident conclusion.)

- Visual inspection of a portion of the precast cable trench and the grades adjacent to the Electrical Equipment Building where the sandbag temporary berm was still in place (to be completed)
- Inclinometers installed along the river bank to identify lateral movement (inclinometers are planned to be installed- Other data and observations are sufficient to reach a confident conclusion)
- Geotechnical borings to determine current soil conditions and capacities (currently not planned- Other data and observations are sufficient to reach a confident conclusion)

5.6.4 Analysis

Identified PFMs were initially reviewed as discussed in Section 3.0. The review considered the preliminary information available from OPPD data files and from initial walk-down observations. Eleven PFMs associated with five different Triggering Mechanisms were determined to be "non-credible" for all Priority 1 Structures, as discussed in Section 3.6. The remaining PFMs were carried forward as "credible." After the design review for each structure, the structure observations, and the results of available geotechnical, geophysical, and survey data were analyzed, a number of CPFMs were ruled out as discussed in Section 5.6.4.1. The CPFMs carried forward for detailed assessment are discussed in Section 5.6.4.2.

5.6.4.1 Potential Failure Modes Ruled Out Prior to the Completion of the Detailed Assessment

The ruled-out CPFMs reside in the Not Significant/High Confidence category and for clarity will not be shown in the Potential for Failure/Confidence matrix.

Triggering Mechanism 2 – Surface Erosion
CPFM 2a – Undermining shallow foundation/slab/surfaces

<u>Reasons for ruling out:</u>

- Slabs were never inundated with floodwater.
- Surface erosion near the ISFSI was not observed during the field assessment.

Triggering Mechanism 2 – Surface Erosion
CPFM 2c – Undermined buried utilities

<u>Reason for ruling out:</u>

- Surface erosion near the ISFSI was not observed during the field assessment.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3a – Undermining and settlement of shallow foundation/slab/surfaces (due to pumping)

<u>Reason for ruling out:</u>

- The basemat and slabs are supported on 10 ft of crushed limestone, which would require higher water velocities to erode than inflow due to pumping can produce.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3c – Undermined buried utilities (due to pumping)

Reason for ruling out:

- Distress that can be attributed to the CPFM was not observed during the field assessments.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3d – Undermining and settlement of shallow foundation/slab (due to river drawdown)

Reason for ruling out:

- The ISFSI is a sufficient distance from the river to be outside the zone of influence for this CPFM.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3f – Undermined buried utilities (due to river drawdown)

Reason for ruling out:

- The ISFSI is a sufficient distance from the river to be outside the zone of influence for this CPFM.

Triggering Mechanism 7 – Soil Collapse (first time wetting)
CPFM 7a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 7b – Displaced structure/broken connections
CPFM 7c – General site settlement

Reasons for ruling out:

- Visual observations during the assessments did not identify settlement of the site during the field assessment.
- Compacted crushed limestone below the basemat, approach slabs, and haul road does not collapse when wetted.
- Site fills were previously wetted. The peak flood elevation prior to 2011 was documented in 1993 as 1003.3 ft, which would indicate that the soils below and surrounding the building had been saturated at this time.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 10b – Displaced structure/broken connections

Reasons for ruling out:

- ISFSI is not subjected to machines or vibrations that could induce liquefaction.
- Liquefaction was not observed at the site during the field assessment.

Triggering Mechanism 11 – Loss of Soil Strength due to Static Liquefaction or Upward Seepage

CPFM 11a – Cracked slab, differential settlement of shallow foundation, loss of structural support

CPFM 11b – Displaced structure/broken connections

Reason for ruling out:

- Liquefaction was not observed at the site during the field assessment.

Triggering Mechanism 12 – Rapid Drawdown

CPFM 12a – River bank slope failure and undermining surrounding structures

CPFM 12b – Lateral spreading

Reasons for ruling out:

- The ISFSI is a sufficient distance from the river to be outside the zone of influence for this PFM.
- Slope failure was not observed at the site.
- River stage level has receded and stabilized as of October 4, 2011.

5.6.5 Results and Conclusions

Possible CPFMs for the ISFSI have been ruled out above. Therefore, there are no CPFMs related to the 2011 flood event that are applicable to the ISFSI.

5.6.6 Recommended Actions

No further actions are recommended for the ISFSI.

5.6.7 Updates Since Revision 0

Revision 0 of this Assessment Report was submitted to OPPD on October 14, 2011. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure. These assessments were incomplete in Revision 0 because the forensic investigation and/or monitoring for most of the Priority 1 Structures was not completed by the submittal date. This revision of this Assessment Report includes the results of additional forensic investigation and monitoring to date for this structure as described below.

5.6.7.1 Additional Data Available

The following additional data were available for the ISFSI for Revisions 1 and 2 of this Assessment Report:

- Results of continued survey by Lamp Rynearson and Associates (see Attachment 6).
- A visual inspection of the cable trench at the grades adjacent to the Electrical Equipment building was observed with no signs of distress.

5.6.7.2 Additional Analysis

The following analysis of additional data was conducted for the ISFSI:

- Results of continued survey by Lamp Rynearson and Associates.

  Survey data to date compared to the original baseline surveys have not exceeded the accuracy range of the surveying equipment. Therefore, deformation at the monitored locations, since the survey baseline was shot, has not occurred.

5.6.7.3 Conclusions

In the assessment of the FCS Structures, the first step was to develop a list of all Triggering Mechanisms and PFMs that could have occurred due to the prolonged inundation of the FCS site during the 2011 Missouri River flood and could have negatively impacted these structures. The next step was to use data from various investigations, including systematic observation of the structures over time, either to eliminate the Triggering Mechanisms and PFMs from the list or to recommend further investigation and/or physical modifications to remove them from the list for any particular structure. Because all CPFMs for the ISFSI have been ruled out, no Triggering Mechanisms and their associated PFMs remain credible for the ISFSI. Therefore, HDR has concluded that the 2011 Missouri River flood did not impact the geotechnical and structural integrity of the ISFSI because the potential for failure of this structure due to the flood is not significant.

Section 5.8

Turbine Building

Omaha Public Power District

5.8 Turbine Building

5.8.1 Summary of Turbine Building

Baseline information for the Turbine Building is provided in Section 2.0, Site History, Description, and Baseline Condition.

The Turbine Building is located within the PA, directly adjacent to the Auxiliary Building to the west, the Maintenance Shop to the north, and the Turbine Building South Switchyard to the south. The Service Building to the east was built integrally with the Turbine Building. The Circulation Water System extends under the Service Building and ties into the bottom of the Turbine Building foundation. The Turbine Building basement floor elevation is established at 990 ft (bottom of floor el. 987.5 ft), while the Service Building floor is established at an elevation of 1007.5 ft. The Service Building is supported by deep foundations. The bottom of the Circulation Water System is at an elevation of 969 ft and is supported by deep foundations. The Auxiliary Building basement floor elevation adjacent to the Turbine Building is established at 989 ft (bottom of mat el. 983.5 ft), and it also is supported by deep foundations. The Turbine Building South Switchyard was constructed at a grade of about 1004.5 ft, and the structures are supported on deep foundations. The Maintenance Shop floor is established at an elevation of 1007.5 ft and is supported on shallow foundations.

The Turbine Building is a multi-floored structure. From the top of the foundation mat at el. 990 ft to el. 1007.5 ft, the structure is cast-in-place reinforced concrete with integral pilasters that align with the steel columns above grade. From el. 1007.5 ft to the roof elevation, the structure consists of braced, rigid steel frames clad with precast concrete sandwich panels. The mat foundation is supported on a combination of 20-in.-diameter Class A steel pipe piles under the turbine generator mat foundation and 12-in.-diameter Class B concrete-filled steel pipe piles under the building mat foundation, all of which are driven to bedrock. Some Class B piles are designated as tension piles and include reinforcing dowels to provide positive tension connection to the foundation mat (see Table 5.8-1).

5.8.2 Inputs/References Supporting the Analysis

Table 5.8-1 lists references provided by OPPD and other documents used to support HDR's analysis.

Table 5.8-1 – References for Turbine Building

Document Title	OPPD Document Number (if applicable)	Date	Page Number(s)
2010 Turbine Building Structure Inspection	SE-PM-AE-1003	7/16/2009	All
Turbine Building 6" and 10" Floor Drain Pipe Breaks	(Summary of CR2009-1365)	Unknown	All
Design Basis Document – Geotechnical	PLDBD-CS-54	Unknown	All
Summary Report of Broken Floor Drain Pipes	NA	3/24/2009	All
Design Basis Document – External Flooding	PLDBD-CS-56	Unknown	All
Work Order Package – 00350972 01 2010 Structural Inspection of the Turbine Building	Reference to Procedure SE-PM-AE-1003	Unknown	All
Naval Facilities Engineering Command, Design Manual 7.01, Soil Mechanics		9/1986	All

Detailed site observations—field reports, field notes, and inspection checklists—for the Turbine Building are provided in Attachment 8.

Observed performance and pertinent background data are as follows:

- The Turbine Building is a Class II structure and is designed to withstand an external hydrostatic load due to flooding of the Missouri River to el. 1007 ft (see PLDBD-CS-56).
- The below-grade structure is independent of the Auxiliary Building, with a 1-ft-9-in. void (expansion joint) between the basement walls. The void is filled with sand below grade and covered with a metal closure plate at ground elevation (see 11405-A-264, Figure 1.2-7 H 12195).
- The Class A piles consist of pipe with 20-in. outside diameter and 1.031-in. wall thickness, which meets API Standard 5L Grade B (F_y = 35 KSI). The piles were driven open-ended to refusal on bedrock. An exploratory boring was drilled through the pipe 15 ft into the bedrock. If a void was encountered, the pile was underreamed and the pile advanced through the void.
- The Class A pile capacities were developed by load testing nonproduction piles for compression, tension, and lateral loads (see PLDBD-CS-54).
- The Class B piles consist of pipe with 12.57-in. outside diameter and 0.25-in. wall thickness, which meets ASTM A252 Grade 2 (F_y = 35 KSI). The piles were driven closed-ended to refusal on bedrock and filled with 4000-psi concrete (see PLDBD-CS-54).
- The soils below the structure were not densified by vibroflotation.
- The structure was protected from floodwaters for the majority of the 2011 flood by an Aqua Dam, combined with sand bags and portable pumps at the exterior overhead door on the south building face adjacent to the Turbine Building South Switchyard. However, the Aqua Dam failed for a short period of time due to being damaged, allowing floodwater to enter the area inside the Aqua Dam perimeter. Approximate river elevation during the period of the breach was 1006 ft.
- Condition report summaries listed in X document many areas of the structure where groundwater has infiltrated the building through previously monitored cracks in the concrete, wall penetrations, and conduit. Observations of the documented groundwater infiltration areas did not identify areas of structural concern.
- General observations of the interior of the structure indicated minor concrete cracking with both current water infiltration (damp to slight running water) and dry walls with signs of water infiltration that has occurred at an earlier time. The observed cracking has been previously recorded and monitored. There were small isolated areas of standing static water in low spots. However, the source of this static water was not found because no water movement was detected.
- Typical wall cracking observed consisted of vertical shrinkage cracks at the horizontal mid-span between pilasters. These are classic concrete shrinkage cracks between the very stiff pilaster elements that occur during the initial concrete curing period.
- The majority of the wall panels encompassed between the pilasters have vertical shrinkage cracks that were either damp to slightly running or show signs of previous water infiltration.
- There is a vertical crack that is full wall height on the north basement wall, approximately 1 ft west of column pilaster TC-9. During additional investigation, it was determined that the crack width at the top of the wall is approximately 0.0625 in. and extends through the thickness of the wall. The crack and the surrounding concrete at the top of the wall were dry with packed dirt/dust within the crack, indicating that the crack had existed long before the flood event.
- The 2010 structural inspection of the Turbine Building (see Reference to Procedure SE-PM-AE-1003) indicated that there was no evidence of significant structural deterioration and that previously installed crack monitors showed no signs of movement.

- The south exterior of the building adjacent to the Turbine Building South Switchyard was visually inspected, and no indications of soil subsidence were observed.
- A column footing in the Maintenance Shop in the first row of footings adjacent to the Turbine Building (Column MG-15) has settled about 2 in., and cracks in the nearby masonry partition walls indicate settlement of the floor slab.
- Below is a summary report of broken floor drain pipes with reference to CR2009-1365:
  - CR2009-1365 was created on March 24, 2009.
  - Two drain lines run parallel to each other: the 6-in. floor drain and the 10-in. waterbox drain. A vendor visually inspected the drain lines because undocumented water was observed draining into the sump pit from both lines. They found a break in the 10-in. drain at the branch tee from the VD-193 drain valve. They could not inspect the 6-in. floor drain because the line does not have a cleanout connection in this area and accessibility through floor drains is restricted by the drain trap at each location.
  - Review of system files shows that a break in the waterbox drain line has been known for quite some time. In 1997, a repair was attempted by core drilling holes in the vicinity of the break and by pressure grouting to seal the pipe, according to the "Water Systems Report Card for Report Period April 1 Through June 30, 1997" (memo PED/EOS SYE-97-123):

    > "Repair of the Turbine Building Basement Drain line header was attempted during this period. The repair procedure consisted of core drilling holes in the vicinity of the leak and pressure grouting to seal the leak. Approximately 10 holes were drilled and it was estimated that a void of approximately 10 by 8 by 1 ft existed under the concrete slab. The void was filled with cement grout but the leak could not be stopped. Boroscope inspection of the pipe exterior performed through the core drills showed considerable pipe damage, in more than one location. The extent of the damage and concern over collapsing the line were determining factors in terminating the pressure grouting operation. FC ECN 97-213 was originated to request that a new drain header be installed."
- The grout was injected in the area by the VD-193 (FW-1A south return box tail valve). At some time later, the Turbine Building sump was cleaned out, and a slab of hardened grout was found in the sump, confirming the grout had flowed through the drain system into the sump. A recent inspection of the floor drains noted a considerable amount of grout in the floor drain south of the FW-3 Condensate Cooler. The drain looks to be almost fully restricted. It seems certain that this grout came from the 1997 effort, indicating that both lines were also broken at that time.
- Review of video taken from the sump on July 22, 2011, and subsequent visual observations indicate groundwater flowing into the sump from all five drain lines.
- OPPD personnel indicated that during an outage, the drain lines that discharge into the sump do not receive flow from the system.
- A majority of the drain lines are located below the mat foundation slab.
- OPPD personnel indicated that the drain lines were cleaned in 2011.

5.8.3 Assessment Methods and Procedures

5.8.3.1 Assessment Procedures Accomplished

Assessments of the Turbine Building included the following:

- Visual inspection of the accessible areas of the interior of the structure from the ground elevation of 1007.5 ft down to the basement floor elevation of 990 ft
- Visual inspection of the exterior of the structure, where accessible

- An assessment of collected survey data to date for indications of trends in the movement of the structure
- A review of previously referenced documents listed in Table 5.8-1.

Additional investigations were performed. These included the following noninvasive geophysical and invasive geotechnical investigations:

- GPR along portions of the basement floor. (Test reports were not available at the time of Revision 0.)
- Seismic surveys (seismic refraction and refraction micro-tremor) in the protected area. (Test reports were not available at the time of Revision 0.)
- TV inspection of the drain pipes below the basement floor. (Test reports were not available at the time of Revision 0.)
- Geotechnical test borings in the protected area. Note that OPPD required vacuum excavation for the first 10 ft of proposed test holes to avoid utility conflicts. Therefore, test reports will not show soil conditions in the upper 10 ft of test boring logs. (Test reports were not available at the time of Revision 0.)

5.8.3.2 Assessment Procedures Not Completed

Assessments of the Turbine Building that were not completed include the following:

- Core holes through the basement floor to measure the size of voids present

5.8.4 Analysis

Identified PFMs were initially reviewed as discussed in Section 3.0. The review considered the preliminary information available from OPPD data files and from initial walk-down observations. Eleven PFMs associated with five different Triggering Mechanisms were determined to be "non-credible" for all Priority 1 Structures, as discussed in Section 3.6. The remaining PFMs were taken into the detailed assessment as "credible." After the design review for each structure, the structure observations and preliminary results of some of the geotechnical, geophysical, and survey data were analyzed, and a number of CPFMs were ruled out as discussed in Section 5.8.4.1. The CPFMs carried forward for detailed assessment are discussed in Section 5.8.4.2.

5.8.4.1 Potential Failure Modes Ruled Out Prior to the Completion of the Detailed Assessment

The ruled-out CPFMs reside in the Not Significant/High Confidence category and for clarity will not be shown in the Potential for Failure/Confidence matrix.

Triggering Mechanism 2 – Surface Erosion
CPFM 2b – Loss of lateral support for pile foundation

<u>Reason for ruling out:</u>

- Surface erosion was not identified near the Turbine Building during the field assessments.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3e – Loss of lateral support for pile foundation (due to river drawdown)

Reason for ruling out:

- The Turbine Building is at sufficient distance from the river and sufficient depth below the ground surface to be outside the zone of influence of the CPFM.

Triggering Mechanism 4 – Hydrostatic Lateral Loading (water loading on structures)
CPFM 4c – Wall failure in flexure
CPFM 4d – Wall failure in shear
CPFM 4e – Excess deflection

Reasons for ruling out:

- The Turbine Building is designed to withstand an external water load due to flooding of the Missouri River to el. 1007 ft (see PLDBD-CS-56). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.
- No signs of structural distress due to lateral loads on the below-grade walls were observed.

Triggering Mechanism 5 – Hydrodynamic Loading
CPFM 5a – Overturning
CPFM 5b – Sliding
CPFM 5c – Wall failure in flexure
CPFM 5d – Wall failure in shear
CPFM 5e – Damage by debris
CPFM 5f – Excess deflection

Reasons for ruling out:

- The Turbine Building is located within the PA and was not subjected to high-velocity river or overland flows capable of producing sufficient hydrodynamic forces.
- No damage from floating debris was observed.
- The Turbine Building is sheltered from high velocity by the Maintenance Building on the north (upstream) side, the Service Building on the east (river) side, and the Auxiliary Building on the west side.

Triggering Mechanism 6 – Buoyancy, Uplift Forces on Structures
CPFM 6a – Fail tension piles
CPFM 6b – Cracked slab, loss of structural support
CPFM 6c – Displaced structure/broken connections

Reasons for ruling out:

- The Turbine Building is designed to withstand an external water load due to flooding of the Missouri River to el. 1007 ft (see PLDBD-CS-56). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis.
- No signs of structural distress due to buoyancy were observed.

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Triggering Mechanism 7 – Soil Collapse (first time wetting)
CPFM 7b – Displaced structure/broken connections
CPFM 7c – General site settlement
CPFM 7d – Piles buckling from down drag

Reason for ruling out:

- The building basement elevation of 990 ft is below the normal river elevation of approximately 992 ft. Therefore, the building foundation system is typically below normal groundwater elevations.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10b – Displaced structure/broken connections

Reasons for ruling out:

- Permanent equipment that has the capacity to produce significant dynamic forces due to vibration is mounted on the base mat foundation slab of the structure. This structure is always below the river level regardless of the flood elevation.
- The turbine was not operated during the flood event.
- This is not a changed condition due to the flood. The Turbine Building has been operating under similar saturated soil conditions and machine vibrations.
- No broken structural connections or structural displacement were observed.
- Liquefaction was not observed to have occurred at the site.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10c – Additional lateral force on below-grade walls

Reasons for ruling out:

- Permanent equipment that has the capacity to produce significant dynamic forces due to vibration is mounted on the base mat foundation slab of the structure. This structure is always below the river level regardless of the flood elevation.
- The turbine was not operated during the flood event.
- This is not a changed condition due to the flood. The Turbine Building has been operating under similar saturated soil conditions and machine vibrations.
- Liquefaction was not observed to have occurred at the site.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10d – Pile/pile group instability

Reasons for ruling out:

- Permanent equipment that has the capacity to produce significant dynamic forces due to vibration is mounted on the base mat foundation slab of the structure. This structure is always below the river level regardless of the flood elevation.
- The turbine was not operated during the flood event.
- This is not a changed condition due to the flood. The Turbine Building has been operating under similar saturated soil conditions and machine vibrations.
- Liquefaction was not observed to have occurred at the site.

Triggering Mechanism 11 – Loss of Soil Strength due to Static Liquefaction or Upward Seepage

CPFM 11b – Displaced structure/broken connections
CPFM 11c – Additional lateral force on below-grade walls
CPFM 11d – Pile/pile group instability

Reason for ruling out:

- Liquefaction was not observed to have occurred at the site.

Triggering Mechanism 12 – Rapid Drawdown

CPFM 12a – River bank slope failure and undermining surrounding structures
CPFM 12b – Lateral spreading

Reason for ruling out:

- The Turbine Building is at sufficient distance from the river and sufficient depth below the ground surface to be outside the zone of influence of the CPFM.

Triggering Mechanism 13 – Submergence

CPFM 13b – Corrosion of structural elements

Reason for ruling out:

- The Turbine Building is designed to withstand an external water load due to flooding of the Missouri River to el. 1007 ft (see PLDBD-CS-56). The peak flood elevation in 2011 was approximately 1006.9 ft, which is less than the structural design basis. Therefore, structural elements being wetted by the 2011 flood were considered in the original design of the facility.

Triggering Mechanism 14 – Frost Effects

CPFM 14a – Heaving, crushing, or displacement

Reason for ruling out:

- The Turbine Building foundation is approximately 20 ft below grade and therefore not frost susceptible. In addition, frost-susceptible connecting utilities are also below frost level.

5.8.4.2 Detailed Assessment of Credible Potential Failure Modes

The following CPFMs are the only CPFMs carried forward for detailed assessment for the Turbine Building as a result of the 2011 flood. This detailed assessment is provided below.

Triggering Mechanism 3 – Subsurface Erosion/Piping

CPFM 3b – Loss of lateral support for pile foundation (due to pumping)

The flow of groundwater into this drain piping system through the breaks in the pipes is one of the Key Distress Indicators discussed in Section 4.

The Turbine Building has a documented history of a void below the foundation dating back to 1997. Conversations with OPPD personnel indicate that groundwater has been flowing at varying rates through these broken pipes into the sump from that time to the present day. The rate of flow into the sump is directly attributable to the hydraulic head of the groundwater because the observed flow rates have increased as the floodwater elevation increased. This drain pipe system was designed as a closed system; therefore, the pipes are not surrounded by appropriate filter systems to preclude the transportation of soils from the surrounding area under the slab. It is logical to assume that as the groundwater flows into the broken piping, the gradient is sufficient to erode the soil.

The Triggering Mechanism and CPFM could then occur as follows: the unfiltered seepage condition will remain until the breaks in the piping system are repaired, which means the potential for further erosion continues unarrested. Erosion could extend out, creating large voids under the Turbine Building mat foundation.

The following table describes observed distress indicators and other data that would increase or decrease the potential for degradation associated with this CPFM for the Turbine Building.

Adverse (Degradation/Direct Floodwater Impact More Likely)	Favorable (Degradation/Direct Floodwater Impact Less Likely)
Previously documented void under the mat foundation	There have been no observed signs of structural distress in the floor slab at the current loading conditions.
Documented breaks in the drain piping below the mat foundation	The dye injected below Column TE15 in the Maintenance Shop was not detected in the Turbine Building sump.
Documented continual groundwater flow from the broken drain piping into the sump	—
The soil around the piling was not compacted to the same requirements as the material under the Class I structures (vibroflotation effort).	—
Column TE15 in the Maintenance Shop has settled over 2 in. (TE column line is adjacent to the Turbine Building)	
Data Gaps: • The size and location of voids below the foundation	

Conclusion

Significance

Potential for Degradation/Direct Floodwater Impact

Indicators for this CPFM have been observed. A void below the mat foundation in the Turbine Building is known to exist, and groundwater is constantly flowing into the sump from all five drain lines. Because the 2011 flood caused increased groundwater flow through the broken drain pipes, the potential that the 2011 flood caused further and more rapid degradation due to this CPFM is high. It is possible that these voids extend beyond the Turbine Building.

Implication

The occurrence of this CPFM would have to be large to negatively impact the capacity of the piling supporting the building. Therefore, the implication of the potential degradation to the Turbine Building for this CPFM is low.

Confidence

This CPFM has two elements: 1) the breaks in the drain pipes allowing groundwater to flow into the sump pit and 2) the potential for voids to develop around the piling system. The flow of groundwater through breaks in the drain pipes has been documented. However, the extent of the associated voids is unknown. The data at hand are not sufficient to rule out this PFM or to conclude that physical modification to ensure that the pilings that support this building have lost capacity because of this CPFM. Therefore, the confidence in the above assessment is low, which means more data are needed to draw a conclusion.

Summary

For CPFM 3b, as discussed above, the potential for degradation is high because of the flow of groundwater through the drain pipes. This degradation would have to be large to impact the integrity or intended function of the structure. The combined consideration of the potential for degradation and the implications of that degradation to a structure of this type puts it in the "significant" category. The data currently collected are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is low, which means more data or continued monitoring and inspections might be necessary to draw a conclusion.

5.8.5 Results and Conclusions

The CPFMs evaluated for the Turbine Building are presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**	CPFM 3b	
Potential for Failure **Not Significant**		

5.8.6 Recommended Actions

The following actions are recommended for the Turbine Building.

Review the GPR data and TV inspection video to assess the impact on the piling system. Further forensic investigations and physical modifications are recommended to address CPFM 3b (Key Distress Indicator #1). These recommendations are described in detail in Section 4.1.3.

Continued monitoring is recommended to include a continuation of the elevation surveys of the previously identified targets on this structure and surrounding site. The purpose is to monitor for signs of structure distress and movement and changes in soil conditions around the structure. The results of this monitoring will be used to increase the confidence in the assessment results. Elevation surveys should be performed weekly for 4 weeks and biweekly until December 31, 2011. At the time of Revision 0, groundwater levels had not yet stabilized to nominal normal levels. Therefore, it is possible that new distress indicators could still develop. If new distress indicators are observed before December 31, 2011, appropriate HDR personnel should be notified immediately to determine whether an immediate inspection or assessment should be conducted. Observation of new distress indicators might result in a modification of the recommendations for this structure.

5.8.7 Updates Since Revision 0

Revision 0 of this Assessment Report was submitted to OPPD on October 14, 2011. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure. These assessments were incomplete in Revision 0 because the forensic investigation and/or monitoring for most of the Priority 1 Structures was not completed by the submittal date. This revision of this Assessment Report includes the results of additional forensic investigation and monitoring to date for this structure as described below.

5.8.7.1 Additional Data Available

The following additional data were available for the Turbine Building for Revisions 1 and 2 of this Assessment Report:

- Results of KDI #1 forensic investigation (see Section 4.1).
- Results of the TV inspection report by Elite Pipeline Services (see Attachment 6).
- Results of geophysical investigation by Geotechnology, Inc. (see Attachment 6).
- Results of geotechnical investigation by Thiele Geotech, Inc. (see Attachment 6).
- Results of continued survey by Lamp Rynearson and Associates (see Attachment 6).

5.8.7.2 Additional Analysis

The following analysis of additional data was conducted for the Turbine Building:

- Results of the TV inspection report by Elite Pipeline Services.

  TV inspections performed in the drain pipes confirmed breaks in the pipes are allowing groundwater to infiltrate the pipes. Additionally, sediment could be observed suspended in the flowing water. During inspection, up to about 3 ft of sand was found in the sump pit.

- Results of geophysical investigation by Geotechnology, Inc.

  GPR tests performed on the Turbine Building floor identified anomalies which could be gravel, soft clay, or possibly voids. Additional ground truthing of the investigation results was performed as part of the KDI #1 forensic investigation.

- Results of geotechnical investigation by Thiele Geotech, Inc.

  All of the SPT and CPT test results conducted for this Assessment Report were compared to similar data from numerous other geotechnical investigations that have been conducted on the FCS site in previous years. This comparison did not identify substantial changes to the soil strength and stiffness over that time period. SPT and CPT test results were not performed in the top 10 feet to protect existing utilities.

- Results of continued survey by Lamp Rynearson and Associates.

  Measurements to date compared to the original baseline measurements have not exceeded the accuracy range of the surveying equipment. Therefore continued deformation at the monitored locations due to the 2011 flood has not occurred.

Additional analysis related to CPFM 3b is discussed in Section 4.1 for KDI #1.

The CPFMs that could not be ruled out in Revision 0 are analyzed below based on the additional data available for Revisions 1 and 2 of this Assessment Report.

Triggering Mechanism 3 – Subsurface Erosion/Piping
CPFM 3b – Loss of lateral support for pile foundation (due to pumping)

CPFM 3b for the Turbine Building is associated with Key Distress Indicator #1. Section 4.1 presents the results of additional forensic investigation that was conducted to ascertain whether the CPFM could be ruled out. The results of the additional forensic investigations show that this CPFM is ruled out. Therefore, assuming that no further concerns are identified through the monitoring program for the Turbine Building (discussed in Section 5.8.6 and continuing until December 31, 2011), the CPFM is moved to the quadrant of the matrix representing "No Further Action Recommended Related to the 2011 Flood."

5.8.7.1 Revised Results

The CPFM evaluated for the Turbine Building are presented in the following matrix, which shows the rating for the significance and the level of confidence in the evaluation:

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**		CPFM 3b

5.8.7.2 Conclusions

In the assessment of the FCS Structures, the first step was to develop a list of all Triggering Mechanisms and PFMs that could have occurred due to the prolonged inundation of the FCS site during the 2011 Missouri River flood and could have negatively impacted these structures. The next step was to use data from various investigations, including systematic observation of the structures over time, either to eliminate the Triggering Mechanisms and PFMs from the list or to recommend further investigation and/or physical modifications to remove them from the list for any particular structure. Because all CPFMs for the Turbine Building other than CPFM 3b had been ruled out prior to Revision 1, and because CPFM 3b has been ruled out by the additional forensic investigations for KDI #1 (see Section 4.1), no Triggering Mechanisms and their associated PFMs will remain credible for the Turbine Building. HDR has concluded that the geotechnical and structural impacts of the 2011 Missouri River flood will be mitigated by the implementation of the physical modifications recommended in this Assessment Report. Therefore, after the implementation of the recommended physical modifications, the potential for failure of this structure due to the flood will not be significant.

Section 5.9

Security Barricaded Ballistic Resistant Enclosures (BBREs)

Omaha Public Power District

5.9 Security Barricaded Ballistic Resistant Enclosures

5.9.1 Summary of Security Barricaded Ballistic Resistant Enclosures

Baseline information for the Security Barricaded Ballistic Resistant Enclosures (BBREs) is provided in Section 2.0, Site History, Description, and Baseline Condition.

Six BBREs are located at the site, as indicated in Figure 5.9-1. Each BBRE consists of a pre-manufactured steel enclosure supported on an elevated reinforced concrete slab. The slab is supported by a 36-in.-diameter reinforced concrete column and a 16-ft-square, 2.5-ft-thick reinforced concrete spread footing. Based on the readily available construction drawings (each structure foundation is assumed to be identical), the foundations were sized based on an allowable soil bearing pressure of 1500 psf.

Prior to the original site development, grades in the area of the BBREs ranged from approximately 990 to 1004 ft. Final site grades, in the area of the BBREs, are established at approximately 1004 to 1005 ft. This would suggest the placement of up to about 15 ft of new fill in some areas. The fill is assumed to consist of a combination of sand, silt, and clay excavated at the site.

5.9.2 Inputs/References Supporting the Analysis

Table 5.9-1 lists references provided by OPPD and other documents used to support HDR's analysis.

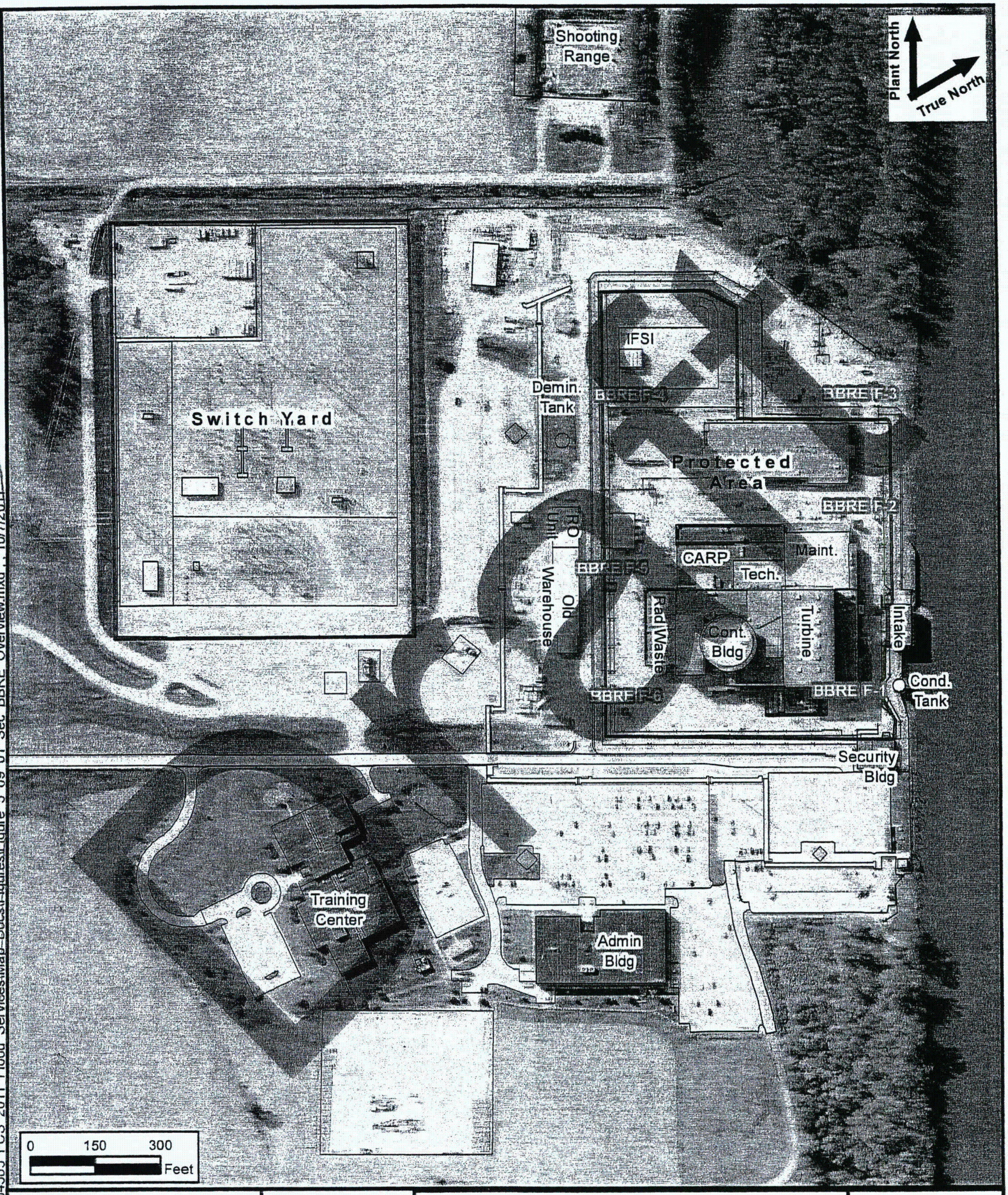
Table 5.9-1 - References for Security BBREs

Document Title	OPPD Document Number (if applicable)	Date	Page Number(s)
Buried Utilities Composite Plan	25036-C-008, Rev. 0 (#60559)	10/8/2004	
Yard Piping Sheet 3	11405-M-314, Sht. 3, Rev. 9 (#10754)	8/13/1973	
Excavation and Grading Building Area	11405-S-272 (#106504)	1/18/1975	
Site Plan Topography	11405-S-251	Unknown	
BBRE #4 Base Details	C-4412 (#62425)	09/06/2007	
(BBRE) Foundation, Column, Platform, Plan, Notes, & Details	145252-SS-S5000 (#62163)	08/04/2006	
Summary of Previous Missouri River Flood Events		Unknown	All
Summary of Previous Groundwater Elevations		Unknown	All
Bathymetric Survey		Unknown	All
Survey Point Elevations		Unknown	All
Naval Facilities Engineering Command, Design Manual 7.01, Soil Mechanics		9/1986	All

Detailed site observations—field reports, field notes, and inspection checklists—for the BBREs are provided in Attachment 8.

Observed performance and pertinent background data are as follows:

- F-1 was constructed with the top of its spread footing flush with the surrounding pavement and is assumed to reside on the pre-existing soils.
- F-2 was constructed with the top of its spread footing ranging from about 1 to 10 in. above the surrounding sloped pavement and is assumed to reside on the pre-existing soils.
- F-3 and F-6 were constructed on top of the pre-existing pavement with reinforcing bars doweled into the pavement.
- F-4 and F-5 were constructed on top of the pre-existing soils.
- Estimated fill placement above original grade at each BBRE is as follows:
  - 10 ft at F-1, F-2, and F-3
  - 1 ft at F-4, F-5, and F-6
- Some site utilities might cross under the BBREs. The Raw Water Piping, with an invert elevation of about 995 ft, has been identified as crossing near or under the foundation for F-1.
- Groundwater was observed flowing into the basement sump of the Turbine Building from floor and condensate drain pipes not designed to intercept groundwater. This condition has a recorded history dating back to 1997.
- Settlement of a column in the Maintenance Shop, north of the Turbine Building, has been documented.
- F-1 and F-2 were protected by an Aqua Dam for the majority of the 2011 flood; however, the Aqua Dam failed for a short period of time due to being damaged, allowing floodwater to enter the area inside the Aqua Dam perimeter. Floodwater overtopped the foundations of F-1 and F-2 during this period. Maximum depth of inundation during the Aqua Dam failure was approximately 2 ft with an approximate river elevation of 1006 ft.
- BBREs F-3 through F-6 are located outside the perimeter of the Aqua Dam, which resulted in the foundations being submerged for a portion of the flood. Maximum depth of flooding was approximately 3 ft above surrounding grades, with an approximate river elevation of 1006.9 ft.
- Water was observed seeping out of the joint between the pavement and the southern face of BBRE F-2's foundation. Estimated flow rate of the seepage appeared to be less than 1 gallon per minute (gpm) on August 25, 2011. OPPD staff had placed sandbags in front of the seeping area. Water appeared clear with no indications of sediment transport. Seepage flow was no longer occurring on September 13, 2011.
- A vertical offset of about 1 in. was identified along the southern side of BBRE F-1's foundation and the top of the surrounding pavement, with the pavement being higher in elevation. The offset could be a pre-existing condition.
- MH-5 is located near F-1 and inside the perimeter of the Aqua Dam. An inflow of water was observed to be flowing through two conduits near the top of the southern wall of the manhole. These conduits connect MH-5 to a manhole outside the perimeter of the Aqua Dam. Observations indicated that up to four pumps had been used, ranging in size from 2 to 4 in.
- Concrete areas in the corridor (paved drive and pedestrian areas between the river and Service Building) have exhibited distress including cracking, settlement, and undermining. Portions of the pavement distress could be pre-existing conditions.
- There is a hole in the pavement and void area beneath the pavement north of the Security Building and east-southeast of MH-5. The hole and void area are outside of the perimeter of the Aqua Dam that surrounded the facility. The pavement failure occurred at the intersection point of pavement jointing. The hole in the pavement is irregular-shaped and is more than 1 ft wide both in the north–south and east–west directions. The void area beneath the hole was approximately 4 ft in diameter by 0.8 ft deep, as measured with a tape measure through the hole.



Z:\Projects\OPPD\164565 FCS 2011 Flood Services\Map Docs\Figures\Figure 5 09 01 Sec BBRE Overview.mxd , 10/7/2011

OPPD Omaha Public Power District	**Security BBRE Overview** **Fort Calhoun Station** Plant and Facility Geotechnical and Structural Assessment	HDR	DATE Oct 2011 FIGURE 5.9-1

- The river bank is armored and has historically protected and stabilized the existing river bank.
- USACE reduced Missouri River Mainstem System releases to 40,000 cfs on October 2, 2011. River levels corresponding to the 40,000 cfs release rate stabilized at FCS on October 4, 2011, at about el. 995 ft.

5.9.3 Assessment Methods and Procedures

5.9.3.1 Assessment Procedures Accomplished

Assessments of the BBREs included the following:

- Visually observed the grade around the perimeter of the structure for erosion and sinkholes
- Visually observed the grade in the vicinity of the structure for cracking or movement
- Probed the grades around the perimeter of the structure for changes in consistency
- Visually observed the structure for indications of movement
- Established survey points on each foundation and periodically monitored
- Assessed survey data to determine whether movement is occurring
- Reviewed previously documented condition reports, as-built plans, site topography, and geotechnical reports to identify possible conditions that could be affected by the flood

Additional investigations were performed to further characterize the subsurface at the facility including areas where conditions indicative of potential flood-related impacts or damage were observed. These included the following non-invasive geophysical and invasive geotechnical investigations.

- GPR in the PA. (Test reports were not available at the time of Revision 0.)
- Seismic surveys (seismic refraction and refraction micro-tremor) in the protected area. (Test reports were not available at the time of Revision 0.)
- Geotechnical test borings in the protected area. Note that OPPD required vacuum excavation for the first 10 ft of proposed test holes to avoid utility conflicts. Therefore, test reports will not show soil conditions in the upper 10 ft of test boring logs. (Test reports were not available at the time of Revision 0.)

5.9.3.2 Assessment Procedures Not Completed

Assessments of the BBREs that were not completed include the following:

- Inclinometers installed along the river bank to identify lateral movement. (Inclinometers are planned to be installed.)

5.9.4 Analysis

Identified PFMs were initially reviewed as discussed in Section 3.0. The review considered the preliminary information available from OPPD data files and from initial walk-down observations. Eleven PFMs associated with five different Triggering Mechanisms were determined to be "non-credible" for all Priority 1 Structures, as discussed in Section 3.6. The remaining PFMs were carried forward as "credible." After the design review for each structure, the structure observations, and the results of available geotechnical, geophysical, and survey data were analyzed, a number of CPFMs were ruled out as discussed in Section 5.9.4.1. The CPFMs carried forward for detailed assessment are discussed in Section 5.9.4.2.

5.9.4.1 Potential Failure Modes Ruled Out Prior to the Completion of the Detailed Assessment

The ruled-out CPFMs reside in the Not Significant/High Confidence category and for clarity will not be shown in the Potential for Failure/Confidence matrix.

Triggering Mechanism 2 – Surface Erosion
CPFM 2a – Undermining shallow foundation/slab/surfaces

Reason for ruling out:

- Surface erosion was not identified near the BBREs during the field assessments.

Triggering Mechanism 5 – Hydrodynamic Loading
CPFM 5a – Overturning
CPFM 5b – Sliding
CPFM 5e – Damage by debris
CPFM 5f – Excess deflection

Reasons for ruling out:

- Sufficient high velocities of the flood water were not identified near the BBREs.
- The structures did not have evident signs of distress identified during the field assessments.

Triggering Mechanism 7 – Soil Collapse (first time wetting)
CPFM 7a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 7b – Displaced structure/broken connections
CPFM 7c – General site settlement

Reasons for ruling out:

- The peak flood elevation prior to 2011 was 1003.3 ft, which occurred in 1993. The peak flood elevation in 2011 was approximately 1006.9 ft. The soils had been previously saturated, and soil conditions were not altered during construction of the BBREs.
- The structures did not have evident signs of distress identified during the field assessments.

Triggering Mechanism 10 – Machine/Vibration-Induced Liquefaction
CPFM 10a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 10b – Displaced structure/broken connections
CPFM 10d – Pile/pile group instability

Reasons for ruling out:

- The structures did not have evident signs of distress identified during the field assessments.
- Machines that induce vibrations are not located near the structures.
- Liquefaction was not observed at the site.

Triggering Mechanism 11 – Loss of Soil Strength due to Static Liquefaction or Upward Seepage

CPFM 11a – Cracked slab, differential settlement of shallow foundation, loss of structural support
CPFM 11b – Displaced structure/broken connections

Reasons for ruling out:

- The structures did not have evident signs of distress identified during the field assessments.
- Liquefaction was not observed at the site.

Triggering Mechanism 12 – Rapid Drawdown

CPFM 12a – River bank slope failure and undermining surrounding structures
CPFM 12b – Lateral spreading

Reasons for ruling out:

- The structures did not have evident signs of distress identified during the field assessments.
- Slope failure was not observed at the site.
- River stage level has receded and stabilized at a level corresponding to the nominal normal river level at 40,000 cfs as of October 4, 2011.

Triggering Mechanism 14 – Frost Effects

CPFM 14a – Heaving, crushing, or displacement

Reason for ruling out:

- The conditions the BBREs will be subjected to during the winter months are not different than what would have occurred before the flood.

5.9.4.2 Detailed Assessment of Credible Potential Failure Modes

The following CPFMs are the only CPFMs carried forward for detailed assessment for the BBREs as a result of the 2011 flood. This detailed assessment is provided below.

Triggering Mechanism 3 – Subsurface Erosion/Piping

CPFM 3a – Undermining and settlement of shallow foundation/slab/surfaces (due to pumping)

Subsurface structures in the general vicinity of the BBREs that were pumped during the flood due to groundwater infiltration included the following:

- Manhole MH-5
- Manhole MH-24
- The Turbine Building sump pit
- The Trenwa near the Security Building
- Turbine Building South Switchyard cable trenches

This CPFM is only considered applicable to F-1 and F-2 because F-3 through F-6 are a substantial distance from the known groundwater pumping locations and would not be in the CPFM's zone of influence.

The Triggering Mechanism and CPFM could then occur as follows: soil deposits could have been carried with the water flow, causing subsurface erosion. If enough soil was removed from these areas, it is possible that portions of the building's foundation and slabs could be undermined.

The following table describes observed distress indicators and other data that would increase or decrease the potential for degradation associated with this CPFM for the BBREs.

Adverse (Degradation/Direct Floodwater Impact More Likely)	Favorable (Degradation/Direct Floodwater Impact Less Likely)
Visual observations of distress in the pavement between F-1 and F-2 indicate some loss of pavement support.	Visual observation of the grades surrounding the BBREs identified grades at normal elevations.
Water was observed seeping out of the joint between the pavement and the west face of the F-2 foundation.	Probing around F-4 and F-5 indicated relatively uniform consistency.
Groundwater was being pumped at locations near the BBREs.	Visual observations did not identify signs of deflection.
—	Survey data to date do not identify measurable movement.
—	Readily available plans indicate that utilities connected to the known locations that were being continually pumped do not cross under the BBREs.
Data Gaps: • The extent of subsurface erosion due to groundwater pumping is not known. • Additional data will be acquired from GPR, seismic survey, and geotechnical test borings. • BBREs F-1, F-2, F-3, and F-6 are surrounded by pavement and could not be probed. • The location of the site utilities could not be confirmed.	

Conclusion

Significance

Potential for Degradation/Direct Floodwater Impact

The CPFM has not been observed at the structures. However, voids created due to groundwater pumping at MH-5 and MH-24 might not have been evident at the time of the field assessments. Additionally, the extent of voids due to pumping of groundwater in the Turbine Building sump has not been determined. Observations of the BBREs indicate the potential that degradation has occurred due to this CPFM is low.

Implication

The occurrence of this CPFM would have to be large to negatively impact the performance of the BBRE foundation. Depending on the location and extent, this would manifest as foundation movement, which could negatively impact the integrity or intended function of the BBREs. Therefore, the implications of the potential degradation for this CPFM is high.

Confidence

The data at hand are not sufficient to rule out this CPFM, nor are they sufficient to lead to a conclusion that the BBRE foundations are or could become undermined because of this CPFM. Therefore, the confidence in the above assessment is low, which means more data are necessary to draw a conclusion.

Summary

For CPFM 3a, as discussed above, the potential for degradation is low. This degradation would have to be large to impact the integrity or intended function of the structures. The combined consideration of the potential for degradation and the implications of that degradation to a structure of this type puts it in the "not significant" category. The data currently collected are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is "low," which means more data or continued monitoring and inspections could be necessary to draw a final conclusion.

Triggering Mechanism 3 – Subsurface Erosion/Piping

CPFM 3d – Undermining and settlement of shallow foundation/slab (due to river drawdown)

The Triggering Mechanism and CPFM could then occur as follows: the drop in elevation of the river is expected to occur at a higher rate than the drop in elevation of the groundwater. This will result in an increased groundwater gradient. This increase could allow for subsurface erosion to occur.

This CPFM is only considered applicable to F-1 through F-3, because F-4 through F-6 are a substantial distance from the river and would not be in the CPFM's zone of influence.

The following table describes observed distress indicators and other data that would increase or decrease the potential for degradation associated with this CPFM for the BBREs.

Adverse (Degradation/Direct Floodwater Impact More Likely)	Favorable (Degradation/Direct Floodwater Impact Less Likely)
—	The structures do not show signs of movement.
—	Survey data to date do not identify measurable movement.
Data Gaps:	
• Additional data will be acquired from GPR, seismic survey, and geotechnical test borings.	

Conclusion

Significance

Potential for Degradation/Direct Floodwater Impact

None of the indicators for the CPFM has been observed at the structures. However, voids due to rapid drawdown might not have been evident at the time of the field assessments. Additionally, the extent of voids created by rapid drawdown could be insignificant. The potential that degradation has occurred due to this CPFM is low.

Implication

The occurrence of this CPFM below a BBRE foundation would have to be large to negatively impact the performance of the BBRE foundations. Depending on the location and extent, this would manifest as foundation movement, which could negatively impact the integrity or intended function of the BBREs. Therefore, the implication of the potential degradation for this CPFM is high.

Confidence

The data at hand are not sufficient to rule out this CPFM nor are they sufficient to lead to a conclusion that the BBRE foundations are or might become undermined because of this CPFM. Therefore, the confidence in the above assessment is low, which means more data are necessary to draw a conclusion.

Summary

For CPFM 3d, as discussed above, the potential for degradation is low. This degradation would have to be large to impact the integrity or intended function of the structures. The combined consideration of the potential for degradation and the implications of that degradation to a structure of this type puts it in the "not significant" category. The data currently collected are not sufficient to rule out this CPFM. Therefore, the confidence in the above assessment is "low," which means more data or continued monitoring and inspections might be necessary to draw a final conclusion.

5.9.5 Results and Conclusions

The CPFMs evaluated for the BBREs are presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**	CPFM 3a CPFM 3d	

5.9.6 Recommended Actions

The following actions are recommended for the Security BBREs.

Review the geotechnical and geophysical data, and assess the impact on the BBREs. Further forensic investigations and physical modifications are recommended to address CPFM 3a and 3d (Key Distress Indicator #1 and #2). These recommendations are described in detail in Section 4.1.3.

Continued monitoring is recommended to include a continuation of elevation surveys of the previously identified targets on this structure and the surrounding site. The purpose is to monitor for signs of structure distress and movement or changes in soil conditions around the structure. The results of this monitoring will be used to increase the confidence in the assessment results. Elevation surveys should be performed weekly for 4 weeks and biweekly until December 31, 2011. At the time of Revision 0, groundwater levels had not yet stabilized to nominal normal levels. Therefore, it is possible that new distress indicators could still develop. If new distress indicators are observed before December 31, 2011, appropriate HDR personnel should be notified immediately to determine whether an immediate inspection or assessment should be conducted. Observation of new distress indicators might result in a modification of the recommendations for this structure.

5.9.7 Updates Since Revision 0

Revision 0 of this Assessment Report was submitted to OPPD on October 14, 2011. Revision 0 presented the results of preliminary assessments for each Priority 1 Structure. These assessments were incomplete in Revision 0 because the forensic investigation and/or monitoring for most of the Priority 1 Structures was not completed by the submittal date. This revision of this Assessment Report includes the results of additional forensic investigation and monitoring to date for this structure as described below.

5.9.7.1 Additional Data Available

The following additional data were available for the BBREs for Revisions 1 and 2 of this Assessment Report :

- Results of KDI #1 forensic investigation (see Section 4.1)
- Results of KDI #2 forensic investigation (see Section 4.2)
- Additional groundwater monitoring well and river stage level data from OPPD.
- Field observations of the river bank (see Section 5.25)
- Results of falling weight deflectometer investigation by American Engineering Testing, Inc. (see Attachment 6).
- Results of geophysical investigation by Geotechnology, Inc. (see Attachment 6).
- Results of geotechnical investigation by Thiele Geotech, Inc. (see Attachment 6).
- Data obtained from inclinometers by Thiele Geotech, Inc. (see Attachment 6).
- Results of continued survey by Lamp Rynearson and Associates (see Attachment 6).

5.9.7.2 Additional Analysis

The following analysis of additional data was conducted for the Security BBREs:

- Groundwater monitoring well and river stage level data from OPPD.

  Data shows that the river and groundwater have returned to nominal normal levels.

- Field observations of river bank

  No significance distress from the 2011 Flood was observed.

- Results of falling weight deflectometer investigation by American Engineering Testing, Inc.

  Falling Weight Deflectometer and associated GPR testing performed in the Paved Access Area identified anomalies such as soft clay and broken pavement. Additional ground truthing of the investigation results were performed as part of the KDI #2 additional investigations.

- Results of geophysical investigation by Geotechnology, Inc.

  Seismic Refraction and Seismic ReMi tests performed around the outside perimeter of the power block as part of KDI #2 identified deep anomalies that could be gravel, soft clay, loose sand, or possibly voids.

- Results of geotechnical investigation by Thiele Geotech, Inc.

  Six test borings were drilled, with continuous sampling of the soil encountered, to ground truth the Geotechnology, Inc. seismic investigation results as part of the KDI #2 forensic investigation. Test bore holes were located to penetrate the deep anomalies identified in the seismic investigation. The test boring data did not show any piping voids or very soft/very loose conditions that might be indicative of subsurface erosion/piping or related material loss or movement.

  All of the SPT and CPT test results conducted for this Assessment Report were compared to similar data from numerous other geotechnical investigations that have been conducted on the FCS site in previous years. This comparison did not identify substantial changes to the soil strength and stiffness over that time period. SPT and CPT test results were not performed in the top 10 feet to protect existing utilities.

  Data from inclinometers to date, compared to the original baseline measurements, have not exceeded the accuracy range of the inclinometers. Therefore, deformation at the monitored locations since the installation of the instrumentation has not occurred.

- Results of continued survey by Lamp Rynearson and Associates.

  Survey data to date compared to the original baseline surveys have not exceeded the accuracy range of the surveying equipment. Therefore, deformation at the monitored locations, since the survey baseline was shot, has not occurred.

Additional analysis related to CPFM 3a is discussed in Section 4.1 for KDI #1, and additional analysis related to CPFM 3d is discussed in Section 4.2 for KDI #2.

The CPFMs that could not be ruled out in Revision 0 are analyzed below based on the additional data available for Revisions 1 and 2 of this Assessment Report.

Triggering Mechanism 3 – Subsurface Erosion/Piping

CPFM 3a – Undermining and settlement of shallow foundation/slab/surfaces (due to pumping)

CPFM 3d – Undermining and settlement of shallow foundation/slab (due to river drawdown)

CPFMs 3a and 3d for the BBREs are associated with Key Distress Indicators #1 and #2. Section 4.1 and 4.2 present the results of additional forensic investigation that was conducted to ascertain whether these CPFMs could be ruled out. The results of the additional forensic investigations show that if the recommendations for physical modifications in KDI #1 are implemented that these CPFMs are ruled out. Therefore, assuming that no further concerns for the BBREs are identified through the monitoring program (discussed in Section 5.9.6 and continuing until December 31, 2011), these CPFMs are moved to the quadrant of the matrix representing "No Further Action Recommended Related to the 2011 Flood."

5.9.7.1 Revised Results

The CPFMs evaluated for the BBREs are presented in the following matrix, which shows the rating for the estimated significance and the level of confidence in the evaluation.

	Low Confidence (Insufficient Data)	High Confidence (Sufficient Data)
Potential for Failure **Significant**		
Potential for Failure **Not Significant**		CPFM 3a CPFM 3d

5.9.7.2 Conclusions

In the assessment of the FCS Structures, the first step was to develop a list of all Triggering Mechanisms and PFMs that could have occurred due to the prolonged inundation of the FCS site during the 2011 Missouri River flood and could have negatively impacted these structures. The next step was to use data from various investigations, including systematic observation of the structures over time, either to eliminate the Triggering Mechanisms and PFMs from the list or to recommend further investigation and/or physical modifications to remove them from the list for any particular structure. Because all CPFMs for the Security BBREs other than CPFMs 3a and 3d had been ruled out prior to Revision 1, and because CPFMs 3a and 3d will be ruled out when the physical modifications recommended for KDI #1 in Section 4.1 are implemented, no Triggering Mechanisms and their associated PFMs will remain credible for the Security BBREs. HDR has concluded that the geotechnical and structural impacts of the 2011 Missouri River flood will be mitigated by the implementation of the physical modifications recommended in this Assessment Report. Therefore, after the implementation of the recommended physical modifications, the potential for failure of this structure due to the flood will not be significant.